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CONSTRUCTIONAL ENGINEERING

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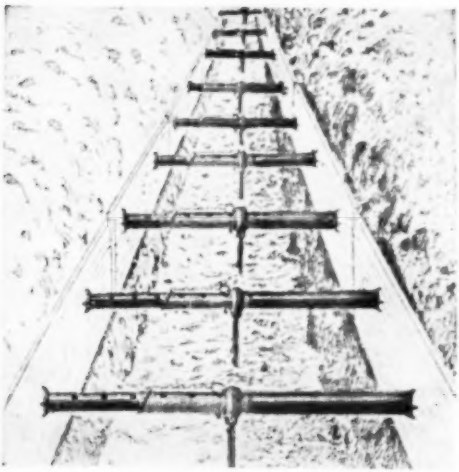

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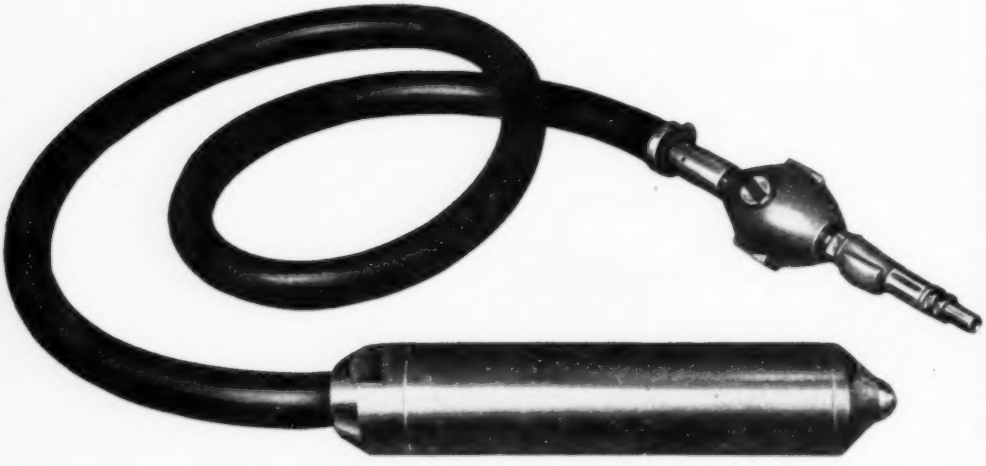

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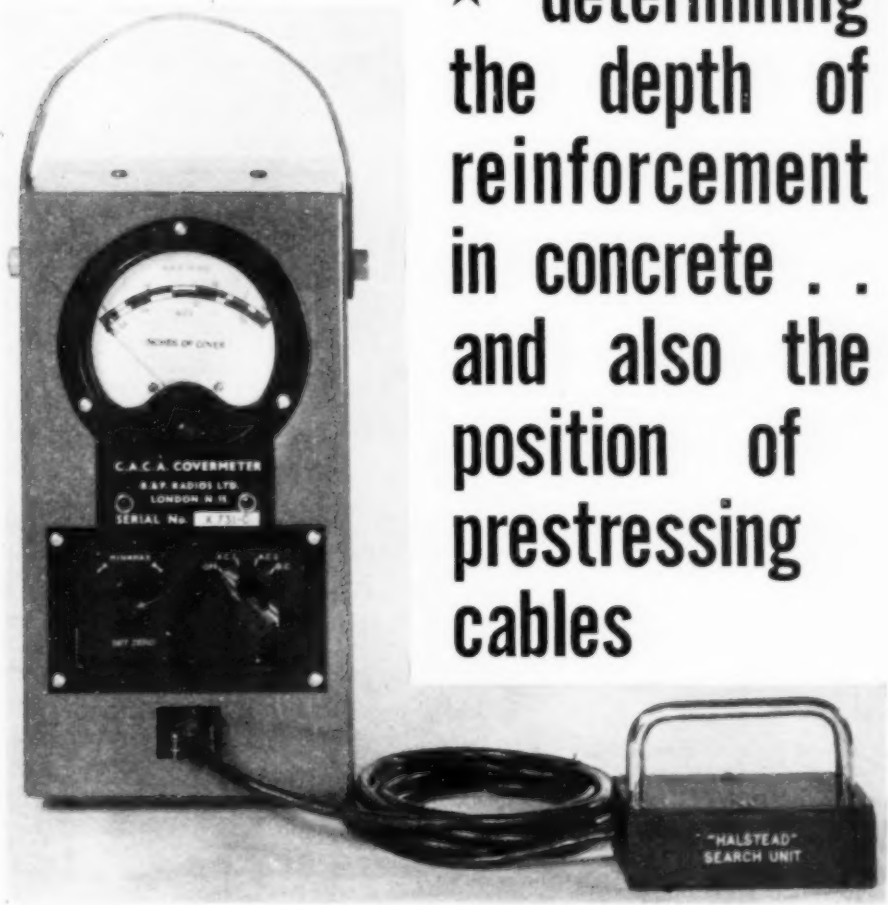

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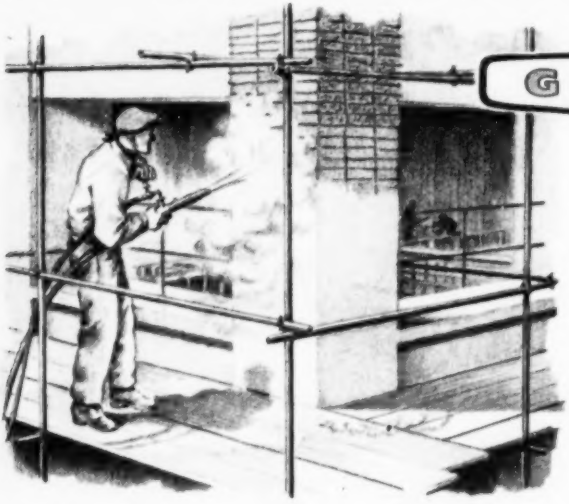

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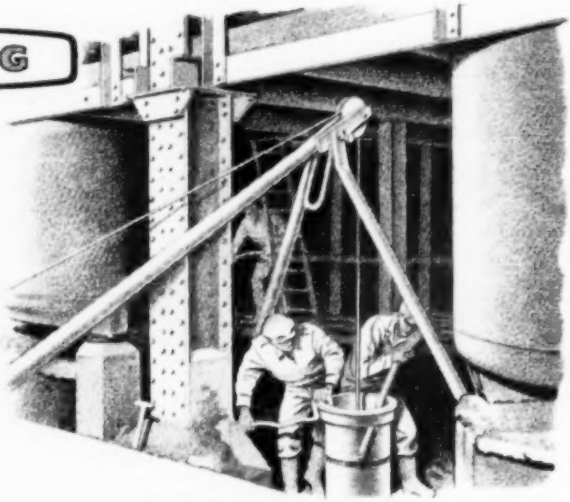

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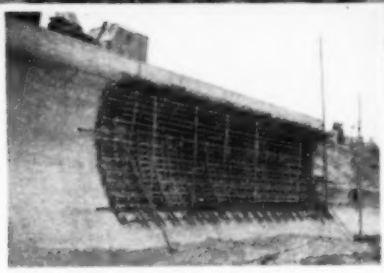

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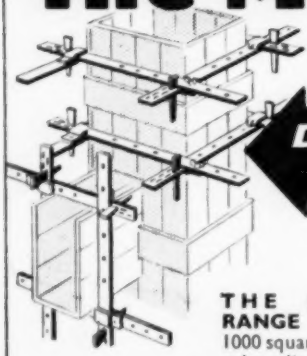

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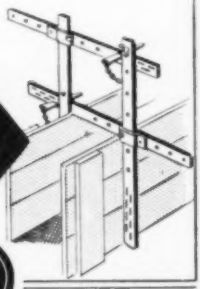

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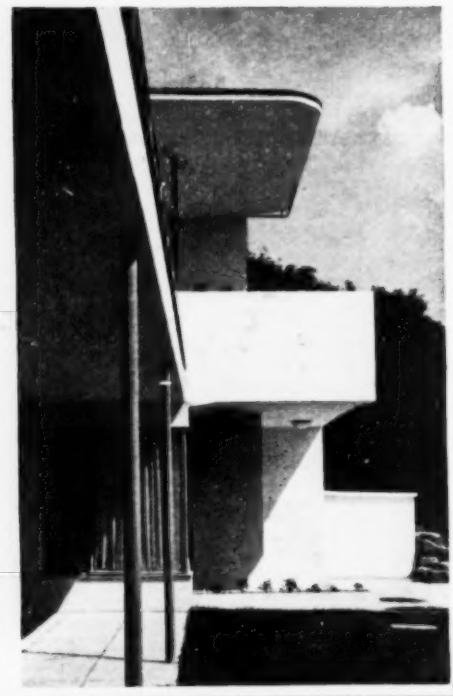

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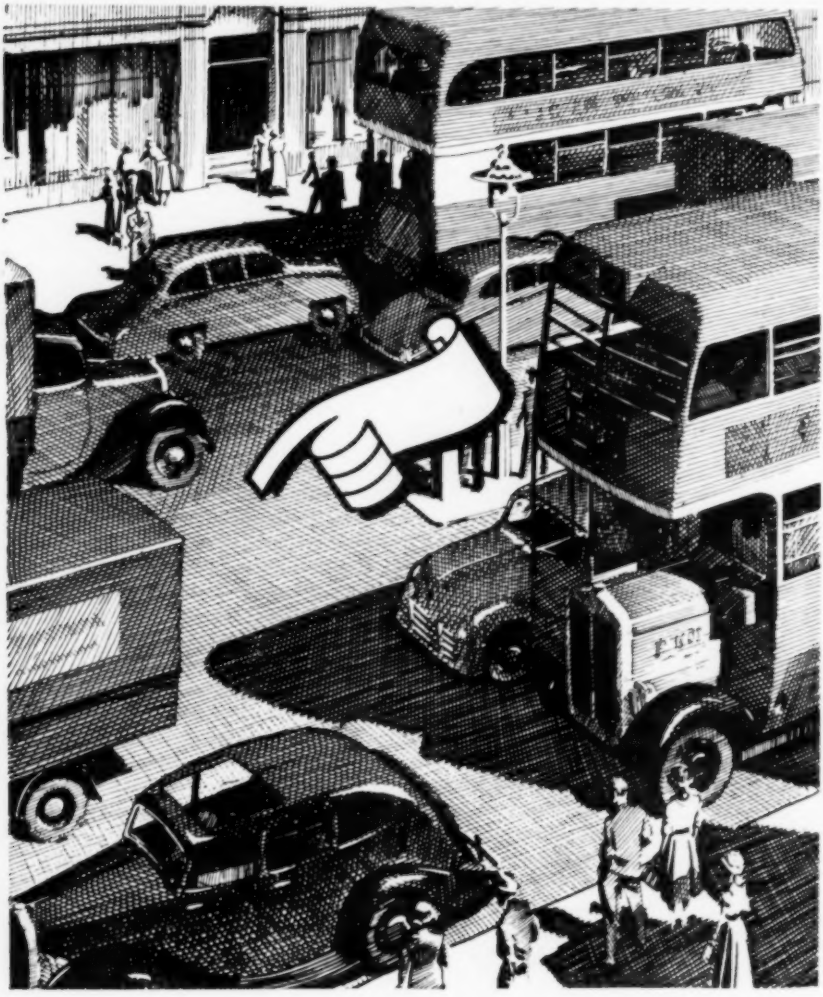

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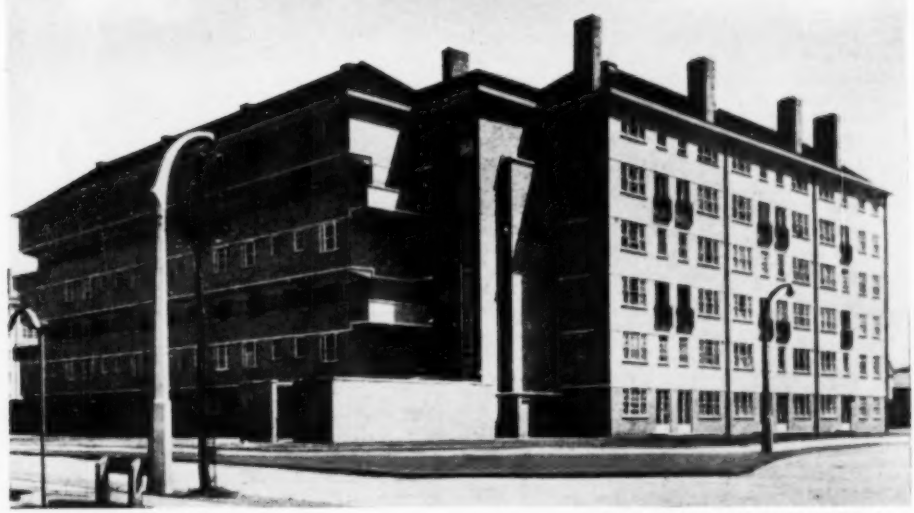

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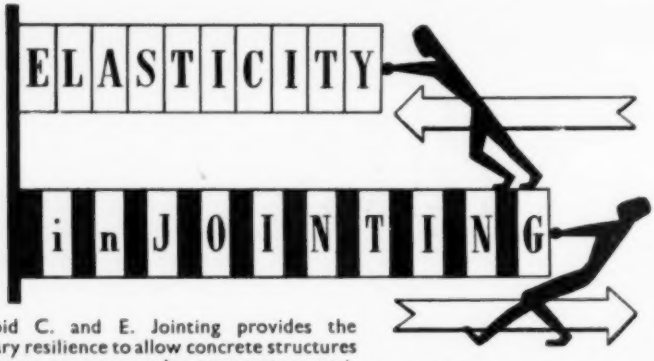

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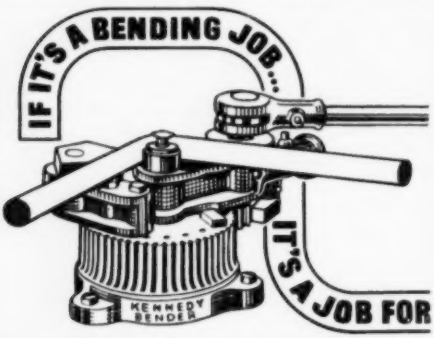

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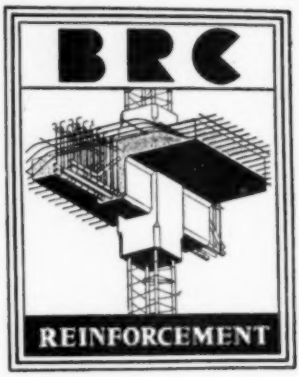

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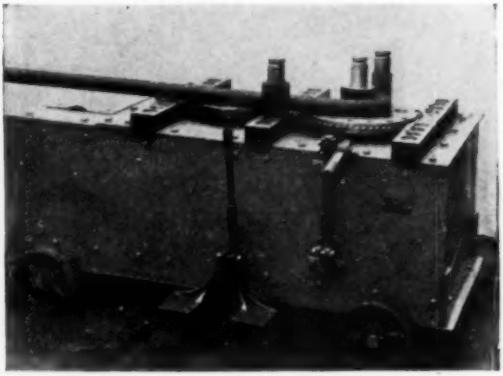

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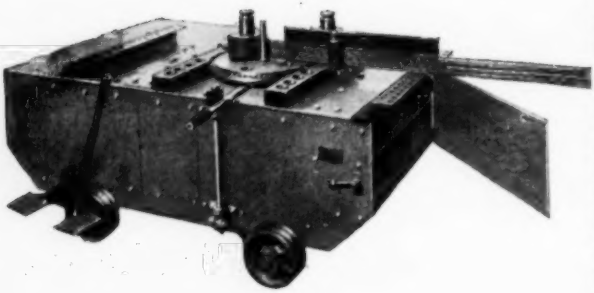

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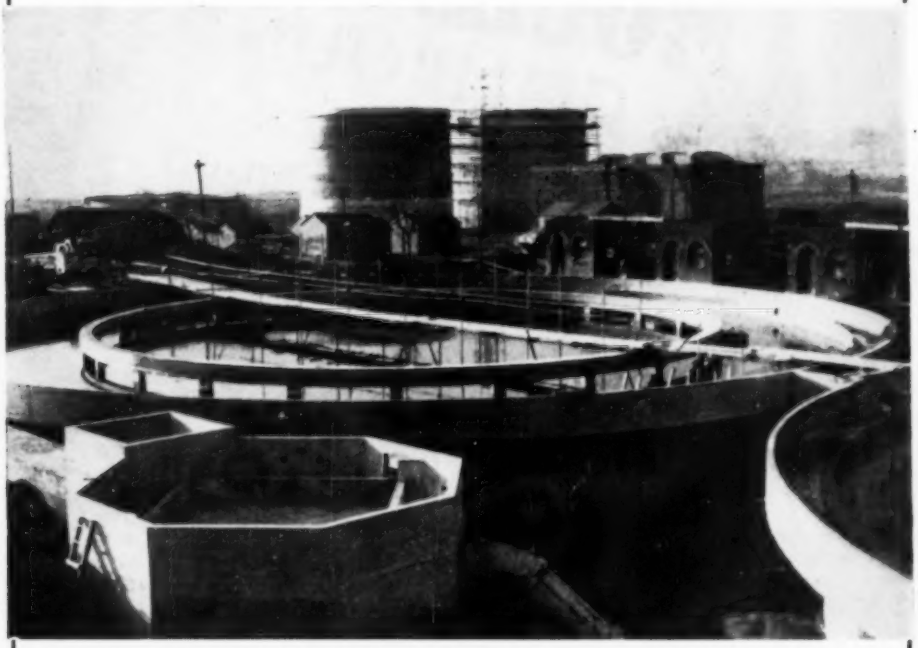

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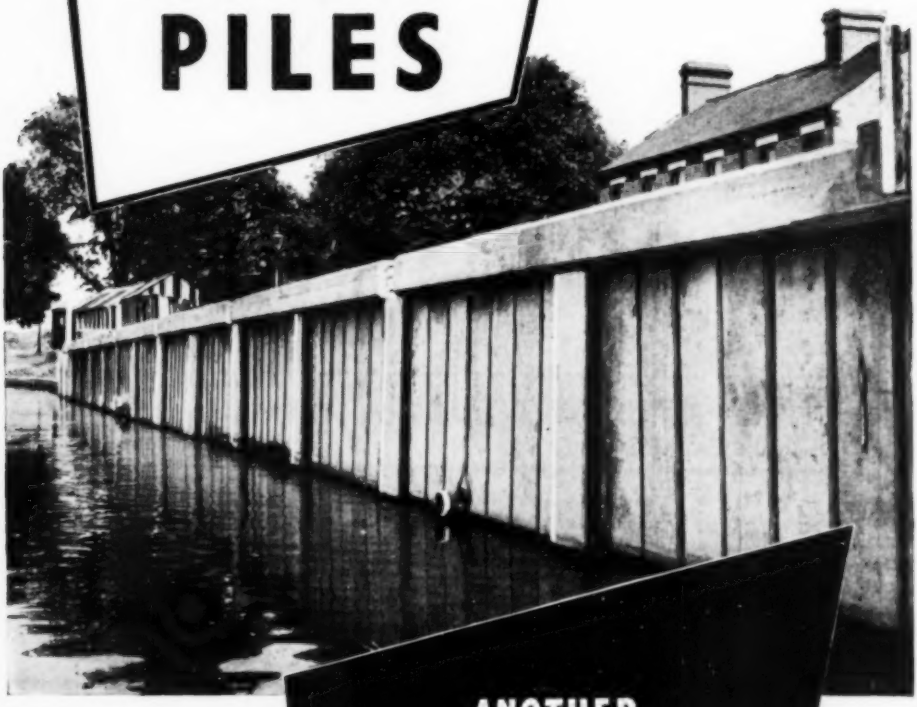

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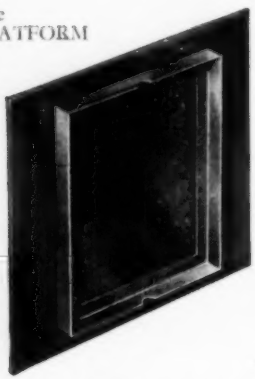

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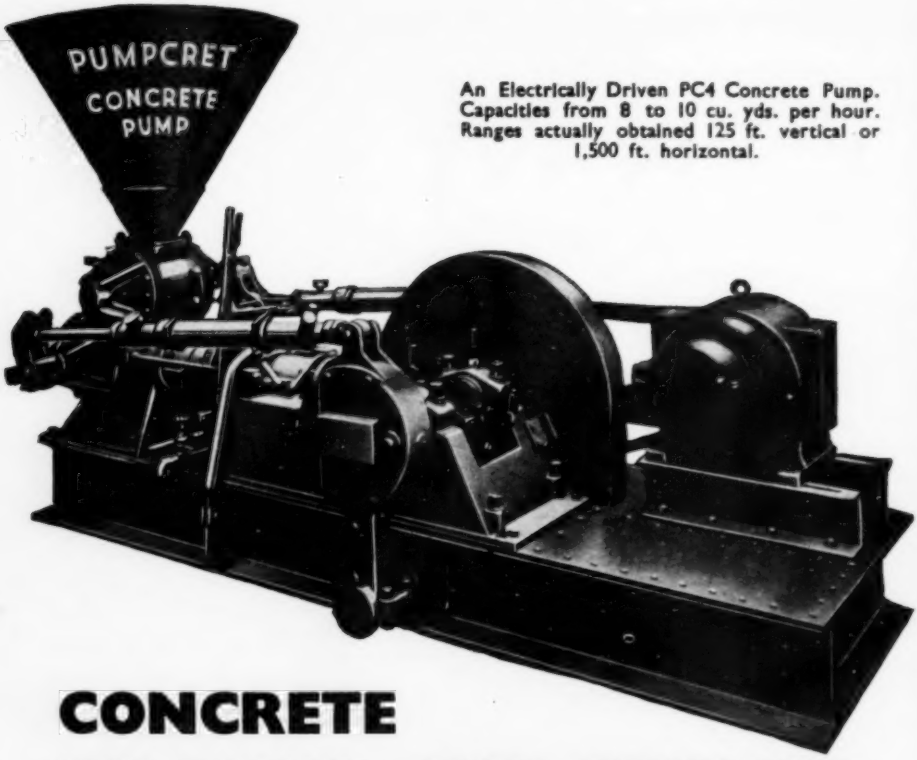

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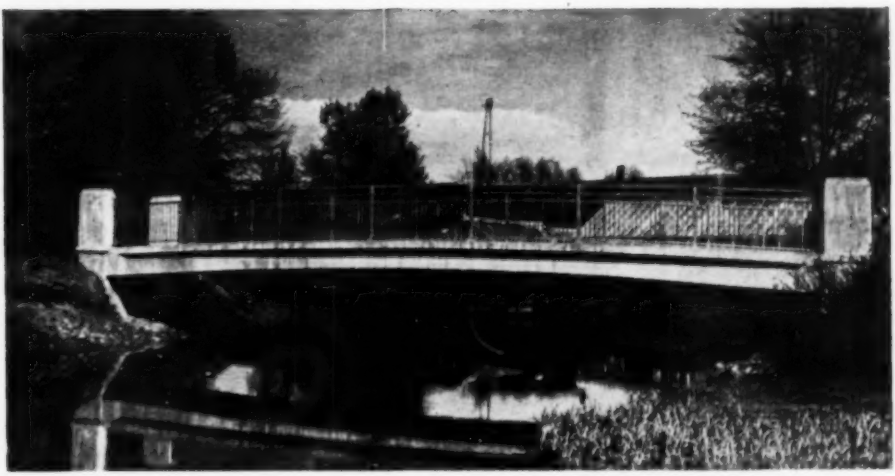

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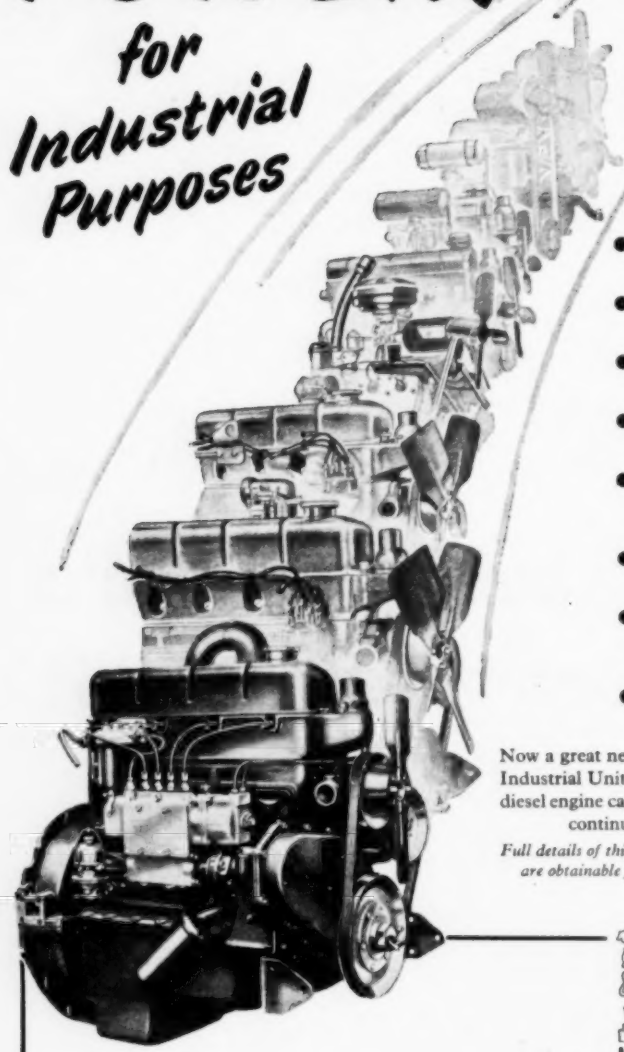

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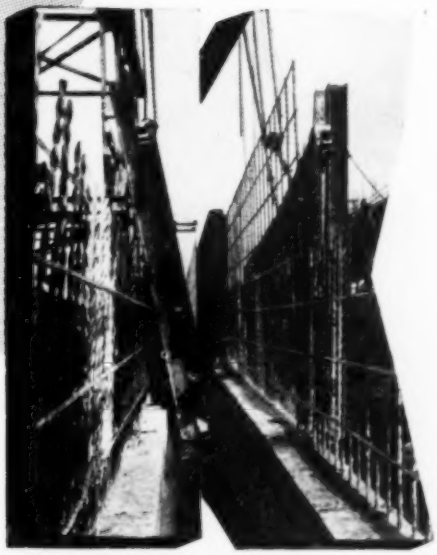

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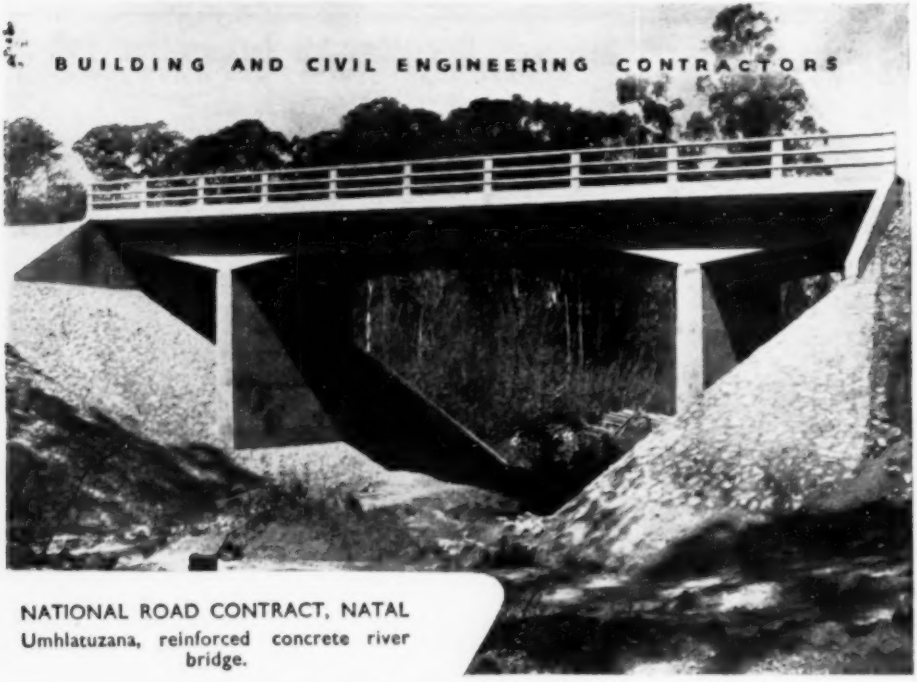

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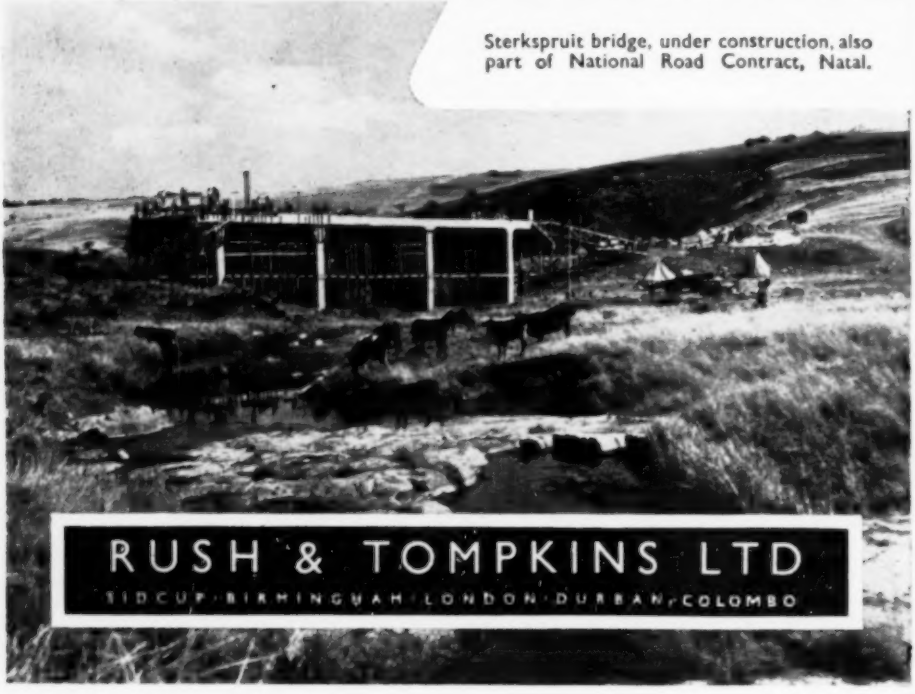

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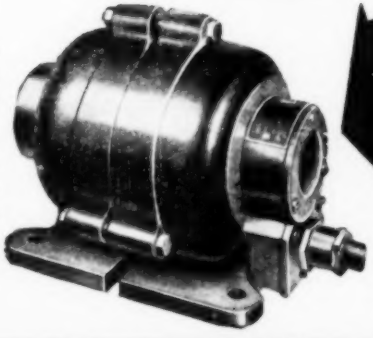

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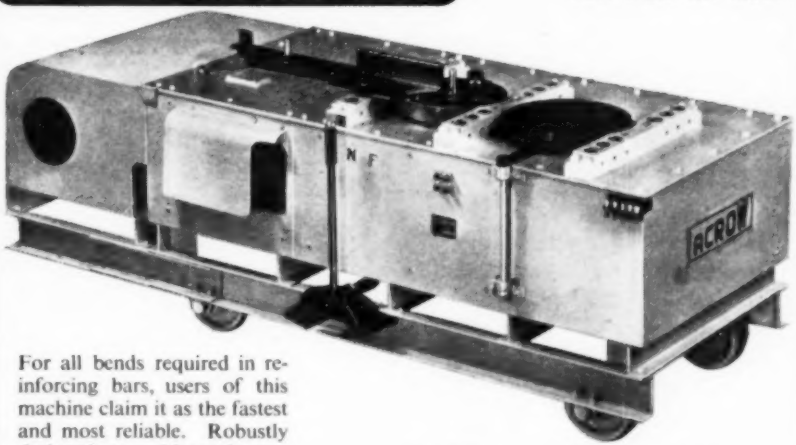

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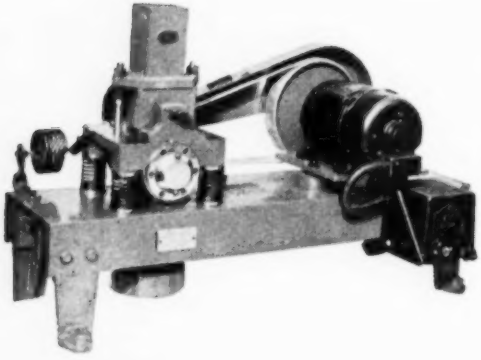

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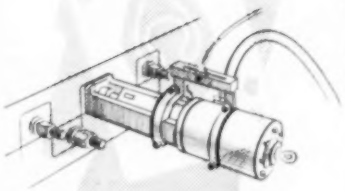

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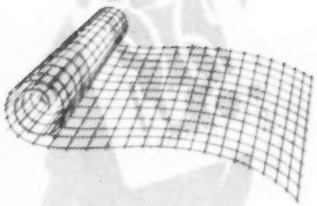

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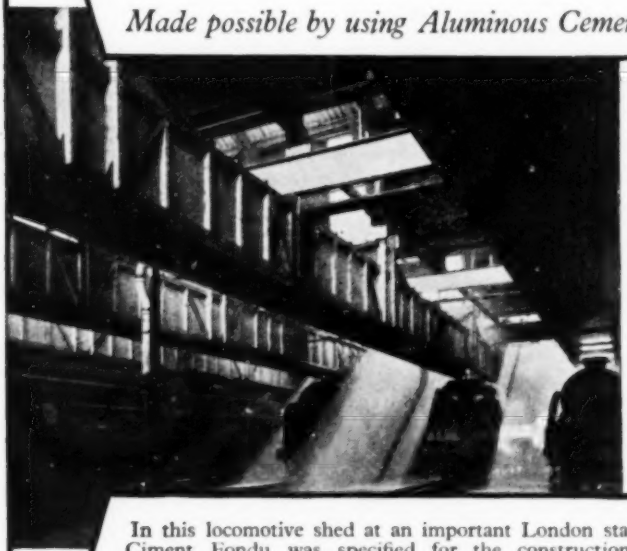

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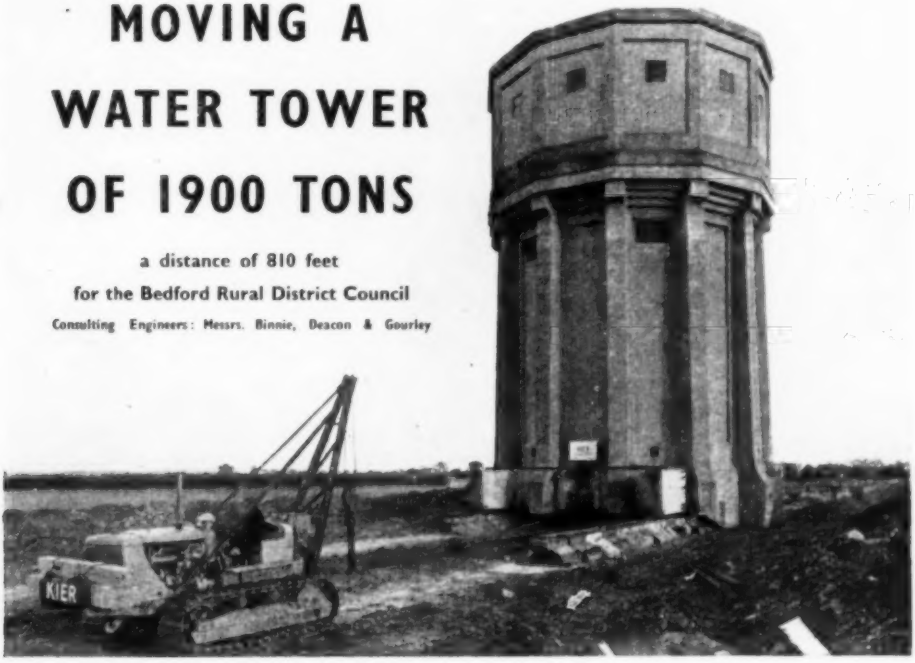

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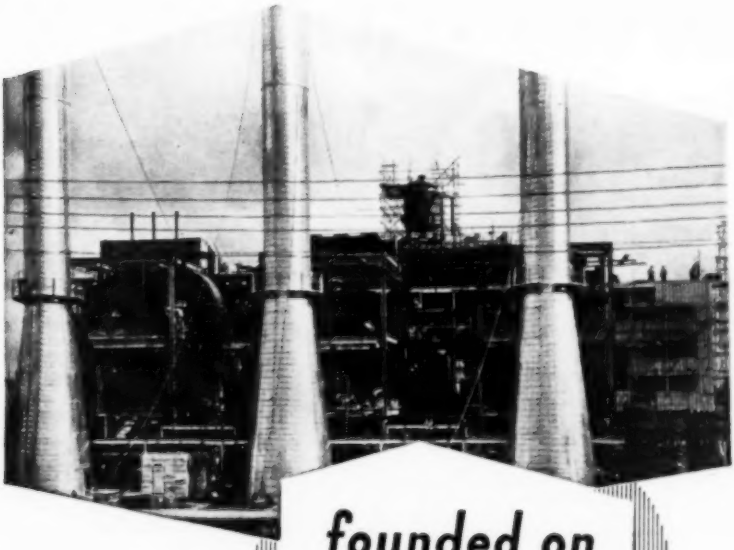

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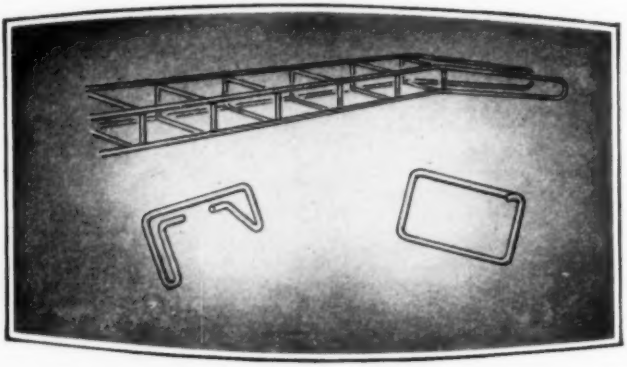

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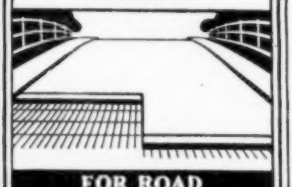

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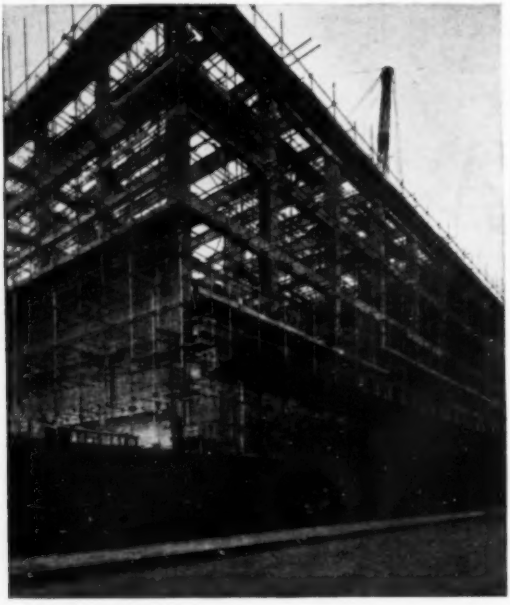

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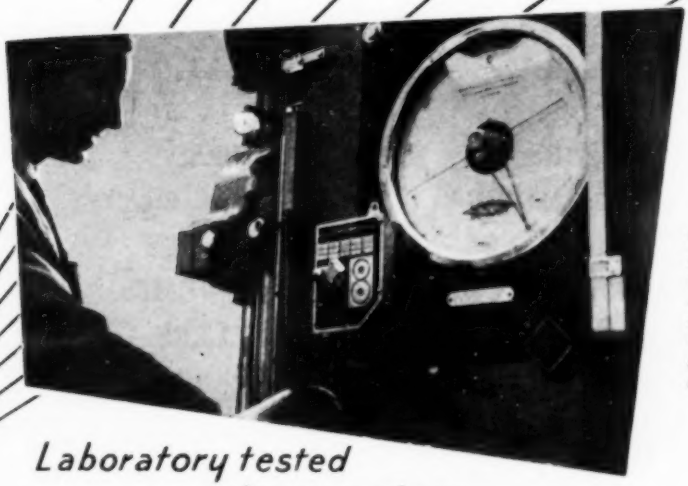

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CONCRETE
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Volume XLVIII, No. 3. LONDON, MARCH, 1953.

EDITORIAL NOTES

Probabilities and Possibilities.

OWNERS of properties that are exceptionally exposed to the forces of nature may well have some anxious thoughts as a result of the flooding last month of large areas of low-lying land washed by the North Sea. Local authorities responsible for coast protection, as well as architects, engineers, and surveyors who advise on the sites and the design of new structures and whose duty it is to ensure that the results of their labours are safe from the elements, will also be perturbed to know that such tragic loss of life and such enormous damage can be done to buildings recently completed at great cost. In the past few years scores of millions of pounds have been spent on the construction of new oil refineries and other important industrial structures on low-lying sites on the estuaries of the Thames and Medway. Yet, before the refineries are in full operation, much damage has been done and production has been stopped because the sites are under several feet of sea-water. Riverside warehouses and factories have been flooded, and part of London's train service was brought to a standstill due to the flooding of a Thames-side power station. Damage has been caused up river beyond central London.

It is natural to ask whether precautions should have been taken to protect these modern and extensive works from the catastrophe that has overtaken them. But the knowledge we acquire after an event can have no part in earlier deliberations. When it was arranged to erect these buildings on their present sites the decisions could be guided only by what was known at the time, and not by what was then the unknown future. Experience has shown that in some places an extra height of only a very few feet on a sea-wall would have kept out the sea, for most of the damage seems to have been caused by the unusually high water flowing over the sea-walls rather than by breaking them down, and it is known that some of the gaps were caused by the scour of the sea on the landward side of the walls after the floods had occurred. It seems that the cost of heightening and strengthening the walls sufficiently to have kept out the sea would not have been great, and would certainly have been less than the cost of repairing the damage that has now been done and other losses due to the flood. But we have to go back four or five hundred years to find any record of a combination of wind and tide causing the sea to rise as high on these shores as it did last month. Even

if this previous flood had been in the minds of those responsible for the structures now flooded, and of the authorities responsible for coast protection, it can hardly be claimed that steps should have been taken to guard against a recurrence of the high seas of five centuries ago during the prospective useful life of the new structures. To have done so would have been beyond the bounds of reasonable prudence, and might with some justice be called a result of superstition rather than a wise precaution. The fact that a natural phenomenon has been known to occur once, and that half a millennium ago, can hardly be considered a compelling reason why prudent people should spend money to protect their property on the assumption that it will happen again during a period in which human beings can expect to enjoy a return for their labour and expenditure. Indeed, floods of the extent of those which have now overwhelmed such vast areas of land are so rare that they could reasonably be considered a risk that could be met by annual insurance premiums rather than by capital expenditure.

Nature, however, is unpredictable. No one can say that the recent floods are the worst of which the sea is capable on these shores, or when a worse will happen. This catastrophe is another reminder of the impotence of man in his struggle to protect himself against the unpredictable vagaries and forces of nature. A few years ago a sea-wall was designed to resist the greatest recorded force of the waves and was sited to be beyond the reach of the highest recorded effect of scour, and yet before it was completed a storm greater than any recorded undermined the wall by scour and moved and damaged it. In the case of the oil refineries and other recent structures now flooded, the previous record of such high water was almost legendary rather than historical in the sense that such data can reasonably be considered by building owners and engineers. Factors of safety are meaningless when no one knows the forces to be resisted. The factors of safety in common use are based on the known strengths of materials, on unvarying laws of mechanics, and on the known effect of the natural phenomenon of gravity which fortunately does not alter; it is interesting to speculate how structures could be built if the force of gravity varied from day to day as much as do the forces of the wind and the waves.

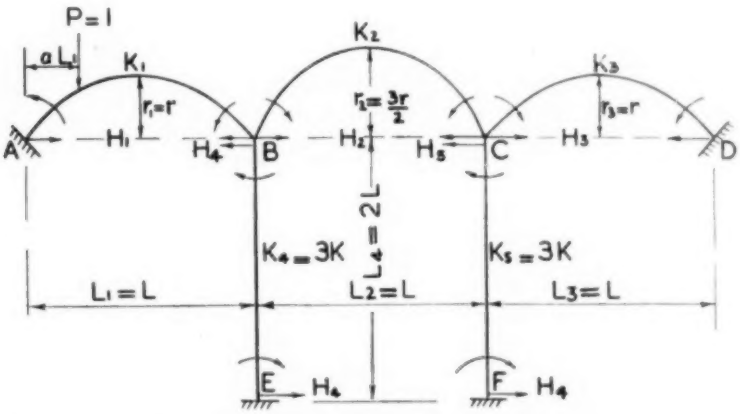
It has been said that one of man's greatest blessings is his inability to foresee the future, and this is no doubt the chief reason why our actions and material thoughts are seldom concerned with posterity. Our labours are mostly concerned with our own well-being during our own lifetime, and if they benefit future generations this result is fortuitous. There were exceptions in the past, such as owners of great estates planting for their heirs and the purse-proud and vain who strove to establish a family and an estate, but there is no opportunity of passing on accumulated riches to direct descendants now that most of such estates are taken in taxation by the state. In the case of the flooded areas there is little doubt that steps will now be taken to protect the countryside and industrial installations against a sea rising to the height experienced last month, to the benefit of the present and future generations. But this work will be done in the full knowledge that a similar or different combination of wind and tide may cause still higher seas in the future with similar or worse floods and destruction, and posterity would not be justified in blaming this generation for making provision against a recurrence of what has happened instead of what might happen.

Analysis of Arches Continuing over Three Spans and Supported on Two Intermediate Columns.

By V. A. MORGAN, M.Eng., A.M.Inst.C.E.

As in the examples in a previous number * the columns of the structure in *Fig.* 1 are assumed to be fixed at their bases, but the arches at the outer springings are fixed. Data are given in the following to enable the influence lines for the horizontal thrusts, bending moments, and shearing forces to be drawn for any section of the arch or column. It is assumed that $K_1 = K_2 = K_3 = K$; $K_4 = K_5 = 3K$, $r_1 = r_3 = L_1 = L_2 = L_3 = L$; $L_4 = L_5 = 2L$; $r_2 = 1{\cdot}5L$; $I_x = I_0 \sec\theta$. The moment-distribution operations are given in *Fig.* 1. The carry-over factors are as in the previous example, but stiffness ratios are modified because of fixity at A and D; also $K_4 = 3K$.

For span AB the fixed-end bending moments FM_A and FM_B are given by formulæ (ii) in the previous article. The resulting sum of the moments at each support decreases in geometrical progression, the common ratio being $\frac{9K_2^2}{d^2}$.



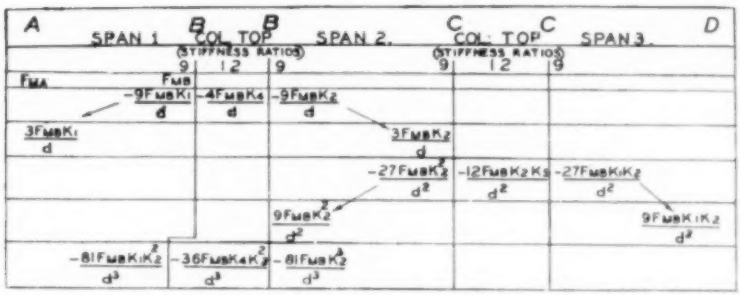


Fig. 1.

* "Concrete and Constructional Engineering", November, 1952.

Hence, since $K_1 = K_2 = K$, and since $d = 30K$,

$$M_{B1} = FM_B\left(1 - \frac{9K_1d}{d^2 - 9K_2{}^2}\right) = \frac{23}{33}FM_B.$$

$$M_A = FM_A + FM_B\frac{3K_1d}{d^2 - 9K_2{}^2} = FM_A + \frac{10}{99}FM_B.$$

$$M_{B4} = -FM_B\frac{4K_4d}{d^2 - 9K_2{}^2} = \frac{-40FM_B}{99}; \quad M_E = \frac{-20FM_B}{99}.$$

$$M_{B2} = -M_{B4} - M_{B1} = -\frac{29}{99}M_B.$$

Similarly it can be shown that

$$M_{C3} = -\frac{FM_B}{33}, \; M_D = \frac{FM_B}{99}, \; M_{C5} = \frac{-4FM_B}{99}, \; M_F = \frac{-2FM_B}{99},$$

and $M_{C2} = \frac{7}{99}FM_B$.

Substituting in formula (i) in the previous article, and writing $Q = \frac{a^2(1-a)}{792}$ from formula (1) in the previous article,

$$H_1 = H_o + \frac{5}{8r}(M_A - M_{B1}) = -\frac{L}{r}(2670 - 2470a)Q.$$

$$H_2 = \frac{5}{8}\cdot\frac{2}{3r}(M_{B2} - M_{C2}) = -\frac{L}{r}(180 - 300a)Q.$$

$$H_3 = \frac{5}{8}\cdot\frac{l}{r}(M_{C3} - M_D) = -\frac{L}{r}(30 - 50a)Q.$$

Since $H_4L_4 + M_{B4} + M_{EB} = 0$ and $H_5L_5 + M_{FC} + M_{C5} = 0$,

$$H_4 = -\frac{M_{B4} + M_E}{L_4} = (360 - 600a)Q. \quad H_5 = -\frac{M_F + M_{C5}}{L_5} = (36 - 60a)Q.$$

There are two unbalanced thrusts, namely, H_B and H_C. The negative signs of H_1, H_2, and H_3 show that these horizontal reactions act inwardly, and that the arrows for H_1 point inwards for any value of a, and for H_2 and H_3 when a is less than 0·6. The positive signs of H_4 and H_5 with a less than 0·6 show that the bending moment due to the thrust is positive and anticlockwise, and that the directions of the arrows are anticlockwise at the top and bottom of the columns.

The unbalanced thrust H_B tends to push joint B to the right and is the sum of the actual values of H_1 and H_4 minus H_2 as indicated by the directions of the arrows in *Fig.* 1, that is,

$$H_B = \left[\frac{L}{r}(2670 - 2470a) + (360 - 600a) - \frac{L}{r}(180 - 300a)\right]Q$$

$$= \left[\frac{L}{r}(2490 - 2170a) + (360 - 600a)\right]Q$$

Similarly, from the directions of the arrows at joint C, $H_C = H_2 + H_5 - H_3$, that is, $H_C = \left[\frac{L}{r}(150 - 250a) + (36 - 60a)\right]Q$.

As the general expression in terms of L and r is unwieldy and unnecessary since an arch of known dimensions would be dealt with in practice, a simple ratio $\frac{L}{r} = 1$ is used in the following, the purpose of which is to show the method of analysis.

Therefore $H_B = (2850 - 2770a)Q$; $H_C = (186 - 310a)Q$.

Deflection due to Unbalanced Forces.

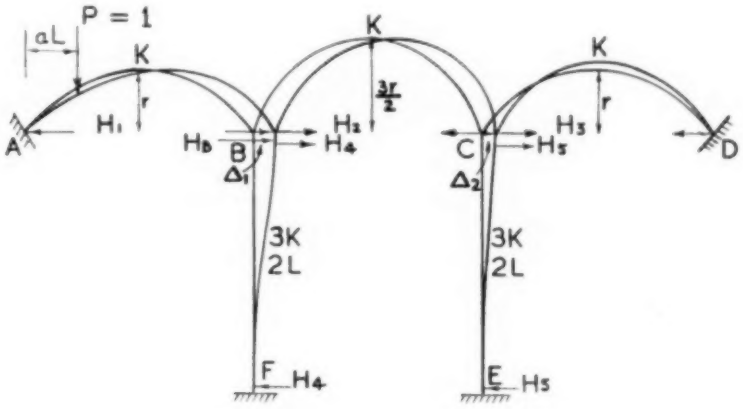
H_B acts at B and produces deflection Δ_1 at B and Δ_2 at C (*Fig. 2*). The fixed-end bending moments at A, E, F, and D are $\frac{15EK\Delta_1}{2r}$, $\frac{6EK_4\Delta_1}{L_4} = \frac{9EK\Delta}{r}$, and $\frac{9EK\Delta_2}{r}$ respectively.

At joint B, $M_{B1} = \frac{-15EK\Delta_1}{2r}$; $M_{B4} = \frac{9EK\Delta_1}{r}$;

$$M_{B3} = \frac{-15EK(\Delta_1 - \Delta_2)}{2r_2} = -\frac{5EK(\Delta_1 - \Delta_2)}{r}.$$

At joint C, $M_{C2} = \frac{5EK(\Delta_1 - \Delta_2)}{r}$; $M_{C3} = -\frac{15}{2}\frac{EK\Delta_2}{r}$; $M_{C5} = \frac{9EK\Delta_2}{r}$. Since $L = r$ and E and K are constant, these terms are omitted from most of the calculations, but r, K, and L are in some cases reinserted to clarify the calculations.

The bending moments are given in *Fig. 2*. These are distributed as in the undeflected case, with the same ratio of decrease but with the additional effects of the bending moments from joint C. Substitute



A		B	B		C	C		D
SPAN 1		COL: TOP	SPAN 2.		COL: TOP	SPAN 3.		
$7.5\Delta_1$	$-7.5\Delta_1$	$9\Delta_1$	$-5(\Delta_1-\Delta_2)$	$5(\Delta_1-\Delta_2)$	$9\Delta_2$	$-7.5\Delta_2$	$7.5\Delta_2$	

Fig. 2.

$$f = \frac{9d}{d^2 - 9K_2^2};\ g = 7{\cdot}5\Delta_1 - 9\Delta_1 + 5(\Delta_1 - \Delta_2);\ h = 7{\cdot}5\Delta_2 - 9\Delta_2 - 5(\Delta_1 - \Delta_2).$$

$$M_{B1} = -7{\cdot}5\Delta_1 + K_1fg + 3K_1K_2\frac{fh}{d} = -\frac{(435\Delta_1 + 93\Delta_2)}{66};$$

$$M_{C3} = -\frac{(435\Delta_2 + 93\Delta_1)}{66}.$$

$$M_{B4} = 9\Delta_1 + \frac{4}{9}K_4fg + \frac{4}{3}K_2K_4\frac{fh}{d} = \frac{(674\Delta_1 - 124\Delta_2)}{66};\ M_{C5} = \frac{674\Delta_2 - 124\Delta_1}{66}$$

$$M_{B2} = -(M_{B1} + M_{B4}) = \frac{217\Delta_2 - 239\Delta_1}{66};\ M_{C2} = -\frac{(239\Delta_2 - 217\Delta_1)}{66}.$$

$$M_A = \frac{15\Delta_1}{2} - \frac{K_1}{3}fg - K_1K_2\frac{fh}{d} = \frac{475\Delta_1 + 31\Delta_2}{66};\ M_D = \frac{475\Delta_2 + 31\Delta_1}{66}.$$

$$M_E = 9\Delta_1 + \frac{2}{9}K_4fg + \frac{2}{3}K_2K_4\frac{fh}{d} = \frac{634\Delta_1 - 62\Delta_2}{66};\ M_F = \frac{634\Delta_2 - 62\Delta_1}{66}.$$

Hence $H_1 = \frac{15K_1\Delta_1}{8r_1^2} + \frac{5}{8r_1}(M_A - M_{B1}) = \frac{5K_1(1108\Delta_1 + 124\Delta_2)}{8 \times 66r^2}.$

$$H_2 = -\frac{15(\Delta_1 - \Delta_2)K_2}{8r_2^2} + \frac{5}{8r_2}(M_{B2} - M_{C2}).$$

Since $r_2 = \frac{3}{2}r$, and $L_4 = L_5 = 2L$, and omitting r, K, etc.,

$$H_2 = \frac{5}{8}\left[\frac{4}{9} \times \frac{3}{L}(\Delta_2 - \Delta_1) + \frac{2}{3}(M_{B2} - M_{C2})\right].$$

Thus $H_2 = -\frac{5 \times 392(\Delta_1 - \Delta_2)}{528L};\ H_3 = -\frac{5(1108\Delta_2 + 124\Delta_1)}{528L};$

$$H_4 = -\frac{M_{B4} + M_E}{L_4} = -\frac{654\Delta_1 - 93\Delta_2}{66L};\ H_5 = -\frac{654\Delta_2 - 93\Delta_1}{66L}.$$

The signs H_2 and H_3 are negative and show that the reactions act inwardly. The negative signs H_4 and H_5 indicate clockwise bending of the columns provided that $654\Delta_2$ is greater than $93\Delta_1$. The directions of the arrows are shown in *Fig.* 2. Therefore joint C is in equilibrium. With positive signs to the forces, $H_2 = H_3 + H_5$. Hence $2084\Delta_1 = 12{,}732\Delta_2$. Therefore $\Delta_1 = \frac{3183\Delta_2}{521}$. The resultant force H_B is $H_1 + H_2 + H_4$; that is,

$$H_B = \frac{1852 \times 121}{3 \times 3183}\left(\frac{\Delta_1}{r}\right) = \frac{1852 \times 121}{3 \times 521}\left(\frac{\Delta_2}{r}\right).$$

Therefore $\frac{\Delta_1}{r} = \frac{9549}{224{,}092}H_B = 0{\cdot}04265H_B.\ \Delta_2 = \frac{1563H_B}{224{,}092} = 0{\cdot}00697H_B.$

Therefore $H_1 = 0{\cdot}455H_B$; $H_2 = 0{\cdot}132H_B$; $H_3 = 0{\cdot}1232H_B$; $H_4 = 0{\cdot}4125H_B$; $H_5 = 0{\cdot}0088H_B$.

Also, substituting for Δ_1 and Δ_2, the following moment-ratios are obtained :

$M_{B1} = -(6{\cdot}6\Delta_1 + 1{\cdot}4\Delta_2)LH_B = -0{\cdot}291LH_B$. $M_{C3} = -0{\cdot}106LH_B$.
$M_{B2} = -0{\cdot}131LH_B$; $M_{C2} = 0{\cdot}1148LH_B$;
$M_{B4} = 0{\cdot}422LH_B$. $M_{C5} = -0{\cdot}009LH_B$; $M_A = 0{\cdot}31LH_B$;
$M_D = 0{\cdot}0702LH_B$. $M_E = 0{\cdot}403H_BL$; $M_F = 0{\cdot}0267LH_B$.

These bending moments and thrust ratios are given in *Fig.* 3.

The unbalanced force H_C is now assumed to act at joint C. From symmetry this action must produce identical ratios of moments and forces, and the signs of the moments are the same. The only change is that H_2 acts outwardly as shown in *Fig.* 4.

Since for unit load on the approach span AB

$$H_B = \frac{a^2(1-a)(2850-2770a)}{792} = Q(2850-2770a),\ H_C = Q(186-310a).$$

Adding the forces for deflected and undeflected conditions the final forces are $H_1 = Q(2670-2470a) - 0{\cdot}455H_B - 0{\cdot}1232H_C = (1349-1170a)Q$ (inward).

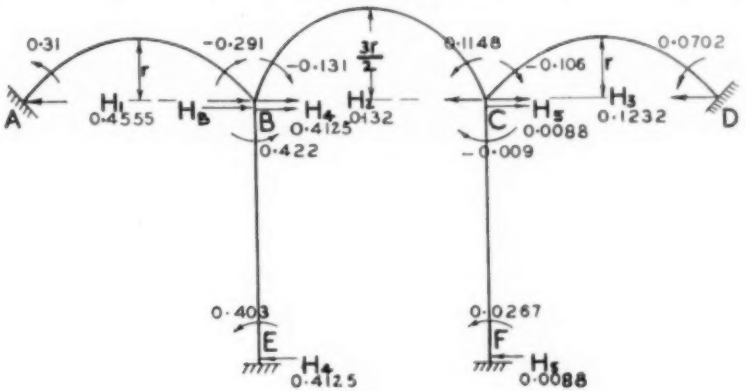


Fig. 3.

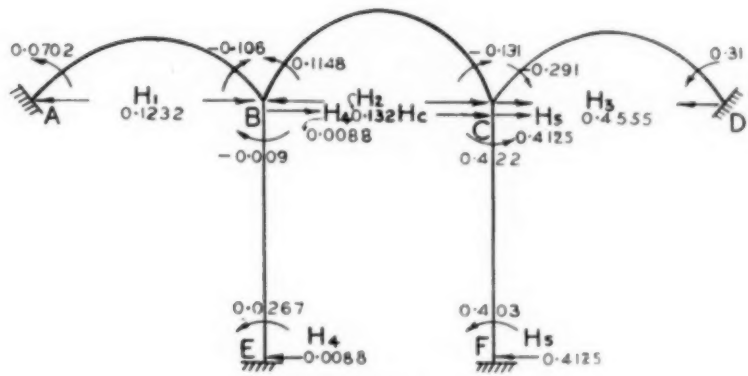


Fig. 4.

$H_2 = Q(180 - 300a) + 0{\cdot}132(H_B - H_C) = (520 - 605a)Q$ (inward between $a = 0$ to $0{\cdot}86$).

$H_3 = Q(30 - 50a) + 0{\cdot}1232H_B + 0{\cdot}455H_C = (465{\cdot}5 - 532a)Q$ (inward from $a = 0$ to $a = 0{\cdot}88$).

$H_4 = Q(360 - 600a) - 0{\cdot}4125H_B - 0{\cdot}0088H_C = -(819 - 546a)Q$ (arrow directions clockwise).

$H_5 = Q(36 - 60a) - 0{\cdot}0088H_B - 0{\cdot}4125H_C = -(66 - 92{\cdot}4a)Q$ (clockwise directions $a = 0$ to $a = 0{\cdot}714$).

For the purpose of illustration, $a = \frac{1}{2}$ is substituted in these equations; then $Q^1 = \frac{1}{6336}$ and $H_1 = 764Q^1$, etc. These ratios are shown in *Fig.* 5, and must be multiplied by Q^1 to obtain actual values.

Summing the moments for the undeflected and deflected cases in accordance with the signs,

$$M_A = FM_A + \frac{10}{99}FM_B + 0{\cdot}31H_BL + 0{\cdot}0702H_CL = FM_A + Q(1016 + 1080a)L$$

$$= \frac{a^2(1-a)}{792}\left(\frac{792}{a} - 1756 + 900a\right)L = QL\left(\frac{792}{a} - 1756 + 900a\right).$$

$$M_{B1} = \frac{23}{33}FM_B - 0{\cdot}291H_BL - 0{\cdot}106H_CL = -QL(21 + 541a).$$

$$M_{B2} = -0{\cdot}131H_BL + 0{\cdot}1148H_C - \frac{29}{99}M_B = -QL(700 - 907{\cdot}4a).$$

In a similar manner M_{B4} and the other moments can be obtained. *Fig.* 5 shows the moment ratios for $a = \frac{1}{2}$, and multiplying by $Q'L$ gives the actual values where $Q' = \frac{1}{6336}$. Thus $M_{B1} = -292Q'L$. It is seen that at joints B and C the moments and forces balance, so that

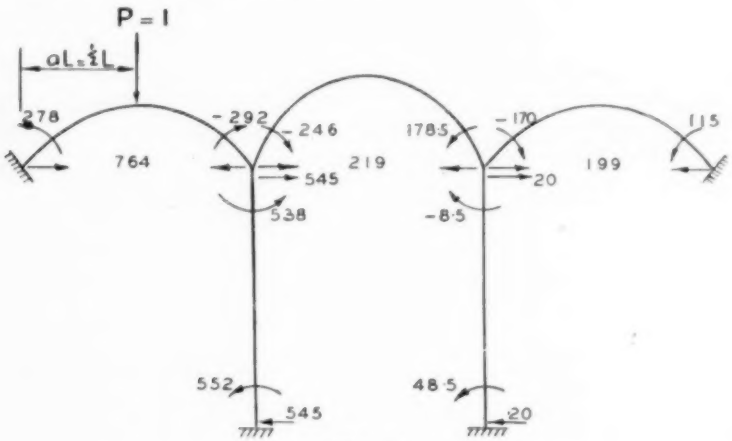


Fig. 5.

$H_A = H_E + H_F + H_D$, $H_{C2} = H_F + H_D$, and $\frac{M_{B4} + M_E}{2L} = -H_4$.

$\frac{M_{C5} + M_F}{2L} = -H_5$, since $L_3 = L_5 = 2L$. These values are a check on the accuracy of the calculations.

The reactions are obtained by calculating moments

$$-R_A L + M_A + M_{B1} + L - aL = 0$$

giving $R_A = 1 - a + \frac{M_A + M_{B1}}{L}$, $R_{B1} = a - \frac{M_A + M_{B1}}{L}$, and $R_{B2} = \frac{M_{B2} + M_{C2}}{L}$

so that

$$R_B = \frac{aL + M_{B2} + M_{C2} - (M_A + M_{B1})}{L} = R_E \text{ (vertical)},$$

$$R_C = R_F \text{ (vertical)} = \frac{M_D + M_{C3} - M_{B2} - M_{C2}}{L},$$

and

$$R_D = -\frac{(M_D + M_{C3})}{L}.$$

Unit Load on Central Span.

As all the ratios due to the unbalanced forces are already obtained, these can be applied by symmetry to obtain the moment and force equations for each span. Thus FM_B is the fixing moment at B due to unit load on the central span, and is $\frac{La}{2}(1-a)^2(2-5a)$. FM_C is the fixing moment at C and is $\frac{La^2}{2}(1-a)(3-5a)$. Then, for the undeflected condition, by moment distribution as before, $d = 9K_1 + 9K_2 + 4K_4 = 30K$.

$$M_{B1} = \frac{-9K_1}{d} FM_B \frac{d^2}{d^2 - 9K_2^2} - \frac{27K_1K_2FM_C}{d^2 - 9k_2^2} = \frac{-9K_1(FM_B d + 3K_2 FM_C)}{d^2 - 9K_2^2}$$

$$= \frac{(-10FM_B + FM_C)}{33} = -\frac{a}{66}(1-a)(20 - 67a + 45a^2)L = -6Q_2S_1,$$

where $Q_2 = \frac{a(1-a)}{396}$ and $S_1 = (20 - 67a + 45a^2)L$.

$$M_{C3} = -\frac{10FM_C + FM_B}{33} = \frac{-a(1-a)}{66}(2 + 23a - 45a^2)L$$

$$= -6Q_2S_2 \text{ where } S_2 = (2 + 23a - 45a^2)L.$$

Similarly $M_{B4} = -8Q_2S_1$; $M_{C5} = -8Q_2S_2$; $M_{B2} = 14Q_2S_1$; $M_{C2} = 14Q_2S_2$; $M_A = 2Q_2S_1$; $M_D = 2Q_2S_2$; $M_E = -4Q_2S_1$; and $M_F = -4Q_2S_2$.

Then, for $A = 0$, $H_1 = \frac{5L}{8r_1}(M_A - M_{B1}) = 5Q_2(20 - 67a + 45a^2)$ acting outwardly for values of a between 0 and 0·413.

$H_2 = H_0 + \frac{5L}{8r^2}(M_{B2} - M_{C2}) = -5Q_2(12 + 138a - 138a^2)$ acting inwardly.

$H_3 = -5Q_2(2 + 23a - 45a^2)$ acting inwardly for values of a between 0 and 0·587.
$H_4 = 6Q_2S_1$ acting anticlockwise for values of a from 0 to 0·413.
$H_5 = 6QS_2$ acting anticlockwise for values of a from 0 to 0·587.

Giving positive signs to H_2 and H_3 (actual values) so that they may be added arithmetically, then the unbalanced force equals the force required to prevent deflection at B, that is

$$H_B = H_2 + H_1 - H_4 \text{ and } H_B = \frac{a(1-a)}{396}(40 + 757a - 735a^2).$$

Also $H_C = H_2 + H_5 - H_3 = \frac{1}{396}a(1-a)(62 + 713a - 735a^2)$. H_C is distributed in the same way as the ratios already worked out and given in *Fig.* 4. H_B is distributed as in *Fig.* 3 but its direction is opposite; thus the moments and forces are of opposite sign and direction to those shown in *Fig.* 3. Hence the final distribution of H_1 is

H_1 (before deflection) $- 0{\cdot}455H_B + 0{\cdot}1232H_C = Q_2(90 - 592a + 469a^2)$.

Similarly $H_2 = -Q_2(46{\cdot}5 + 496a - 469a^2)$, and H_3, H_4, and H_5 can be obtained in the same way.

The final moments are then obtained—

$$M_{B1} = LQ_2(402a - 120 - 270a^2) + 0{\cdot}291H_B - 0{\cdot}106H_C$$
$$= LQ_2(115 - 546{\cdot}5 + 406a^2).$$

Similarly, M_{B2}, etc., are obtained.

By substituting $a = \frac{1}{2}$ and omitting the multiplier $Q_2' = \frac{1}{1584}$ for forces and $Q_2'L$ for moments, the values shown in *Fig.* 7 are obtained. The force equations for span CD loaded are obtained from symmetry, measuring aL from the springing D, or, if it is desired to measure aL from the springing C, the horizontal force

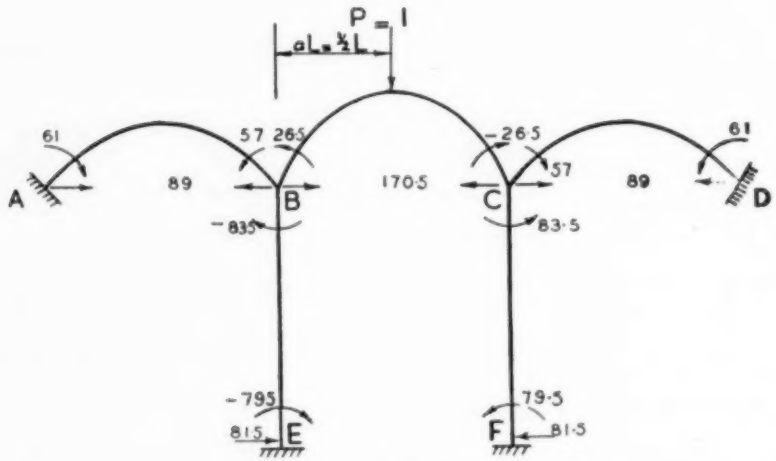


Fig. 6.

equations are obtained directly by substituting $1 - a$ for a in the equations already obtained for span AB loaded.

The moment equations also are obtained by substituting $1 - a$ for a in the equations for span AB loaded, but the signs will be opposite since the directions of the moments are reversed; the directions of the thrust at E and F also are opposite for CD loaded compared with AB loaded. Thus all possible vertical loads on the span can be used and, from the influence lines, the forces and moments determined.

For any other span-rise and stiffnesses of the arch and columns due to end-fixity the moment and force influence-line formulæ can be derived, so that all types of three-span parabolic ($I_0 \sec \theta$ variation) arches and column lengths both for symmetrical and unsymmetrical frames can be analysed.

Solution of Arch Frames. Variation of Shape.

To obtain influence lines for arch ribs varying as

$$C \sec \theta = I_0\left[1 + B\left(\frac{x}{L} - \frac{1}{2}\right)^2\right] \sec \theta$$

for any value other than $B = 0$ (previously dealt with) the spans should be divided into an arbitrary number of equal parts placing a unit load at each part $a = 0{\cdot}1$, $0{\cdot}2$, $0{\cdot}3$, etc., in turn. The procedure is similar to that shown for the $B = 0$ variation. Stiffness ratios S and carry-over factors R corresponding to the values of B for the arch are obtained from *Fig.* 12 in the November, 1947, number of this journal.

If one springing is hinged, the stiffness-ratio of the other springing can be obtained from S_1 (other springing hinged) $= S(1 - R^2)$. The moments M_A and M_B are obtained for the value of B and the particular value of a such as $a = 0{\cdot}1$ from *Fig.* 6 in the October, 1947, number of this journal, by interpolation or by the more accurate method of drawing an influence line as in *Fig.* 8 in that number. M_B is the value corresponding to $1 - a$. The moments are distributed by using

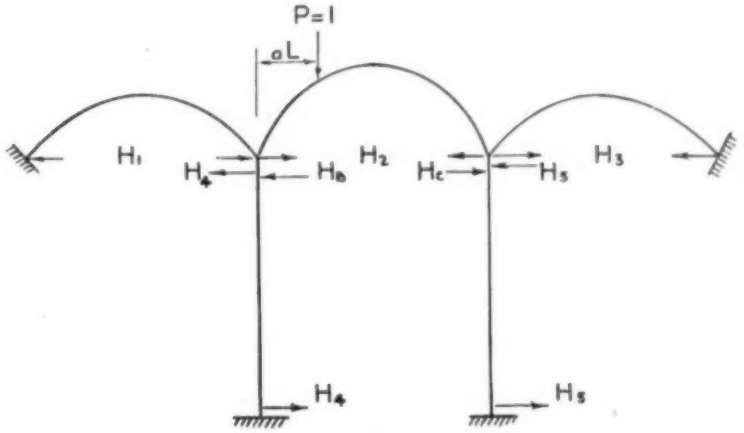


Fig. 7.

the carry-over factors R and stiffness ratios determined by the end-fixity. As the moments are distributed they will decrease in geometrical progression, and they can be summed to infinity as previously shown. The horizontal thrusts can be determined from

$$H_1 = H_0 + G(M_A - M_B) \quad . \quad . \quad . \quad . \quad (A)$$

H_2, etc., can be determined in the same manner. H_0 (the hinged-springing condition) is obtained by interpolation or by an influence line from *Fig.* 2 of the November, 1947, number. The value of G is obtained from *Fig.* 2 of the previous article. The values H_B and H_C, the unbalanced forces at B and C, are then determined. These values all correspond to a value such as $a = 0{\cdot}1$.

In a similar manner values of M_A, M_B, etc., H_1, H_2, H_3, etc., are determined corresponding to $a = 0{\cdot}2$ and so on for the other values of the arch span. These determine the moments, etc., for the undeflected condition of the arch frame.

Deflected Condition.

The fixed-end moments for the tops and bases of the columns are calculated as already described unless the column section varies, in which case they can be calculated from stiffness ratios and carry-over factors determined from unit rotation moments as in the previous article. The fixed-arch springing-moment values are obtained as in *Fig* 10 in the November, 1947, number. The moments are distributed and summed in a similar manner, using the same stiffness ratios and carry-over factors as for the undeflected condition. The horizontal forces are determined from $H = H_o + G(M_A - M_B)$, etc. In this case H_o is due to the movement Δ_H and is obtained from *Fig.* 10 of the November, 1947, number for a two-pin arch. From the horizontal forces, Δ_1 is obtained in terms of Δ_2, and thence H_1, etc., and the moment ratios are obtained in terms of H_B. In a similar manner H_1, etc., and the moment ratios are determined in terms of H_C.

Substituting the values of H_B and H_C obtained from the undeflected condition corresponding to $a = 0{\cdot}1$ and adding, according to the signs and the directions of the arrows, the moments and forces due to the deflected and undeflected conditions, the final moments and forces corresponding to $a = 0{\cdot}1$ are obtained. Similarly moments and forces corresponding to $a = 0{\cdot}2$, etc., are obtained, and the complete influence lines may be drawn for all variations of arch spans and columns.

As the horizontal forces and moments for the deflected condition are obtained as ratios of H_B and H_C, they will be constant for all values of H_B and H_C, and the work involved in obtaining them will be very little greater than is required for the $B = 0$ condition.

For the undeflected condition, the values of M_A, M_B, and H_D corresponding to $a = 0{\cdot}1$, etc., will be simple arithmetical values; the calculations are not lengthy and can be tabulated for each separate value of a. In fact the whole procedure of summation of deflected and undeflected conditions can be reduced to simple arithmetical tabular form, deriving the necessary data from the graphs.

Having obtained the moments and thrusts at the springings and the ends of the columns, the frames became statically determinate and reactions and forces at all sections can be obtained by moments. The moment-distribution method can be used for any number of continuing spans, the number of unbalanced

thrusts H_B, H_C, H_D, etc., being derived in terms of Δ_1, Δ_2, Δ_3, etc. There is, however, the advantage that there is one less equation to be solved for Δ than the number of unbalanced thrusts H_B, etc. The values of Δ are then resolved in terms of H_B, etc., and the final forces and moments are derived using the same procedure as has been described.

The Strength of Concrete at the Time of Loading.

MR. NIELS M. PLUM, Director of Research of the Danish National Institute of Building Research and Chairman of the Concrete and Reinforced Concrete Section of the Danish Institution of Civil Engineers, has sent the following comment on the suggestion made in the Editorial Notes in this journal for November, 1952, and January, 1953.

" I am not sure that maximum loading always occurs after 28 days. In Denmark it has been observed frequently that building materials for the upper floors are stored on the lower floors to such an extent that the load is two or three times as great as that assumed in the calculations.

" In the original suggestion and in the discussion it has been assumed that the concrete is steadily gaining in strength for quite a while. This assumption may not be correct in many circumstances. When concrete is stored under ideal conditions, as assumed, the strength will increase and ultimately be about 50 per cent. greater than the strength at 28 days. In practice the storage conditions are rather imperfect. For example, moist-curing may be discontinued early, so that an ultimate strength only about 10 per cent. greater than the strength at 28 days may be reached at three months, and the strength of the member may be further diminished by the continuous application of the load and by chemical and other attacks so that in some cases the strength may, at later ages, be low. In some cases it may be lower than that upon which the permissible stress is based.

" In principle the establishing of permissible stresses in a building material, as for instance concrete, is not primarily technical but economic. The greatest economy in the cost of a building is obtained when the initial cost plus the capitalized cost of maintenance over the whole life of the building is a minimum. We have made a study of this question in Denmark with special reference to concrete floors and we have found that the least total cost is obtained when the factor of safety is lowered from the now traditional figure of 4 to about 2·5. Similar studies may be undertaken for any other kind of structure. As will be realized, our results so far seem to indicate that the permissible stresses may be increased, but I think that the increases should be based mainly on economic considerations such as the one described."

[The possibility of construction loads exceeding the working load must be foreseen and guarded against whether the design stresses are based upon the application of the working load at one month or three months; this was mentioned in our Editorial Note for January of this year. Adequate control of the construction would prevent loss of strength due to inadequate curing; the possible reductions due to creep and to attack by aggressive agents would always have to be considered.—ED.]

Lectures on Concrete.

THE following lectures have been arranged by the Ministry of Works. Admission free.

Essentials of Good Concreting, by E. H. MacMillen. Municipal Technical College (Building Department Annexe), Hopwood Lane, Halifax. March 16, 7.30 p.m.

Lightweight Concrete, by T. Whitaker. Y.M.C.A., Connaught Hall, Blackett Street, Newcastle-upon-Tyne. March 17. 7.0 p.m.

Essentials of Good Concreting, by E. Woodwark. Maidstone Technical College, Tonbridge Road, Maidstone. March 18. 7.0 p.m.

Concrete Placing and Formwork, by L. J. Murdock. School of Building, North Staffs. Technical College, Couldon Place, Stoke-on-Trent. March 25. 7.15 p.m.

Book Reviews.

"Publications of the International Association for Bridge and Structural Engineering." Vol. XII. (Zurich: The Association. 1952. No price stated.)

THIS volume contains seventeen papers, including in the English language: "Partially Prestressed Concrete Constructions built in the Eastern Region of British Railways, 1948–1952," by P. W. Abeles; "The Design of Small Railway Underbridges, with special reference to Erection and Maintenance under Traffic," by P. S. A. Berridge; "Approximations following from the Maximum Condition applied in Theories of Plasticity and Earth Pressure," by H. Craemer; "Shear Deflections in Latticed Structures," by C. D. Crosthwaite; "Statics of the Vierendeel Girder," by I. A. el Demirdash; "Model Verification of the classical Flutter Theory as adapted to the Suspension Bridge," by F. B. Farquharson; "The Fabrication and Erection of the Dome of Discovery," by J. D. Vaughan; and "Mathematical Prediction of Suspension Bridge Behaviour in Wind from Dynamic Section Model Tests," by G. S. Vincent. In the French language with summaries in English are: "Corrosion Problems of Concrete," by P. Chevrier and R. Houbas; "Tests Concerning Creep and Shrinkage Losses in Prestressed Concrete," by G. Dawance; "The Sectional Method in Slab Theory," by R. L'Hermite; and "Altering the Levels of the Supports of Continuous Slabs to reduce the Stresses in the Concrete in Constructions combining Structural Steel and Concrete," by M. Oudotte and M. Guerin. In the German language with summaries in English: "Continuous Cylindrical Beams of Prismatic Cross Section," by E. Gruber; "The Two-dimensional Problem in the case of Periodically Altering Temperature Effects," by P. Lardy; "Shear Centre and Torsion," by F. Stussi; and "The Influence of Frictional Forces and Bearing Pressures in connection with Prestressed Concrete," by W. Swida.

The paper by MM. Chevrier and Houbas deals with the effects of pure water, salt water, and frost on concrete hydraulic structures in France. Methods are suggested for resisting the attacks of the salt water and frost, but the problem of combating attack by pure water is stated to be still far from being solved.

The design of bridges comprising a structural steel frame with a concrete deck has been a subject of research for many years to determine how much allowance can be made of the combined action of the two materials. Where the beams are continuous the problem is more difficult because of the tendency of the slab to crack over the supports. This difficulty is claimed to be overcome by the adoption of the method described by MM. Oudotte and Guerin and used in the construction of three bridges in France. To prevent the slab cracking, a compressive stress is produced in it by raising the inner supports of the continuous beams before casting the slabs and, after the concrete has hardened, lowering the beams to their original level. The slabs are bonded to the steel beams by means of bars, in the form of helices, welded to the flanges of the beams and embedded in the concrete. Tests carried out over about three years show that the method was successful in preventing cracking, and that there was a considerable reduction in deflections.

In "Continuous Cylindrical Beams of Prismatic Cross Section" Dr. Gruber derives a method of design for prismatic thin slab structures continuing over several supports which, it is claimed, is the first accurate presentation of the subject.

The book is an interesting addition to the literature of structural engineering and it is a pity that occasional slips in the translations from the French and German languages into English mar an otherwise excellent publication.

"Reactions Between Aggregates and Cement." National Building Studies Research Paper No. 14. By F. E. Jones. (London: H.M. Stationery Office. 1952. Price 1*s.* 3*d.*)

IN parts of the U.S.A. the disintegration of concrete has been traced to the reaction between Portland cement and some types of stone used as aggregate. This paper is an account of the research on this subject carried out in the U.S.A., and methods are suggested for testing aggregates to ascertain if they are reactive. Opal, chalcedony, tridymite, cristobalite, phyllites, and some types of volcanic rocks have been found to be reactive and to be a source of danger. Although the problem is not important in Great Britain, the paper suggests that some deposits of aggregates may be reactive.

Test of Prestressed Concrete Stairs.

THE stairs to the upper floors of Plashet County Secondary School for Girls at East Ham are of prestressed precast concrete, comprising flights, 20 ft. 7 in. long by 2 ft. 10 in. wide of ten steps, together with the upper and lower landings. The landings are 9⅛ in. thick, each step has a rise of 6 in. and a tread of 11 in., and the total rise is 5 ft. The waist of the steps is 8 in. Two of these flights side by side form a stairway 5 ft. 8 in. wide. *Fig.* 1 shows a flight after testing, and *Fig.* 3 gives the details; provision is made for fixings for the wooden treads and balusters. A total of 54 flights was required.

The stairs are prestressed with "Macalloy" high-tensile steel bars, post-tensioned to 42 tons per square inch. The bars were greased, wrapped with a spiral layer of one-ply roofing felt, and fixed in the mould prior to casting. Three cranked bars of 1 in. diameter pass through the lower landing and through the inclined waist, and four bars, ⅞-in. diameter, pass through the upper landings. The nuts at the ends of the bars bear on mild steel plates 1½ in. thick. The concrete immediately adjacent to the plates is reinforced with mild steel bars (*Fig.* 3).

Fig. 4 shows the timber mould ready to receive the concrete, which was a 1 : 1·5 : 3·4 mixture with a maximum size of aggregate of ¾ in. The water-cement ratio was 0·4. Crushing tests on 6-in. cubes gave the following strengths in lb. per square inch: at seven days, 5500; at 14 days, 6150; at 35 days, 6650. The concrete was vibrated by immersion vibrators and by vibrators attached to the mould. The mould (*Fig.* 4) was stripped the day after casting, and *Fig.* 5 shows a flight ready for tensioning with the jack in position.

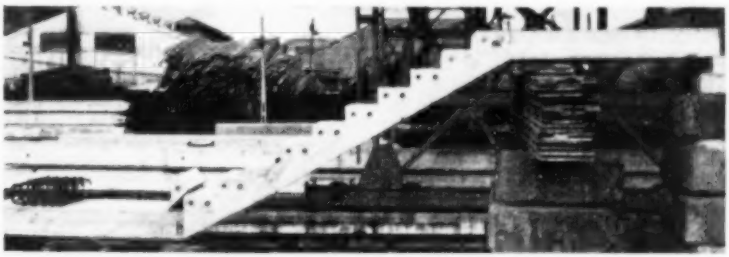


Fig. 1.—Recovery of Staircase after Removal of Load.

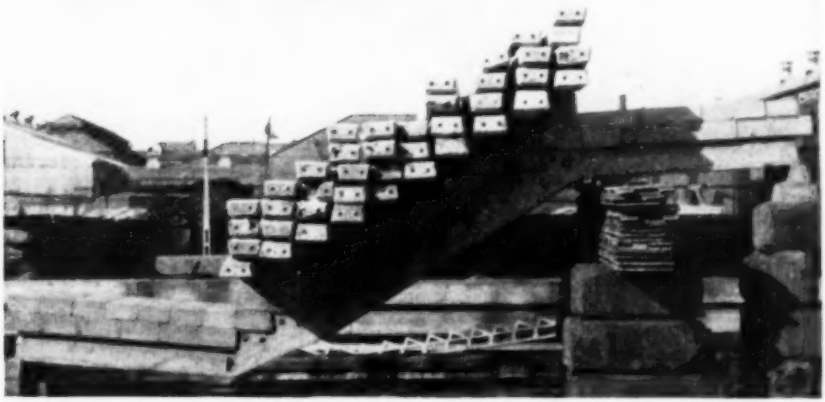


Fig. 2.—Load at Failure.

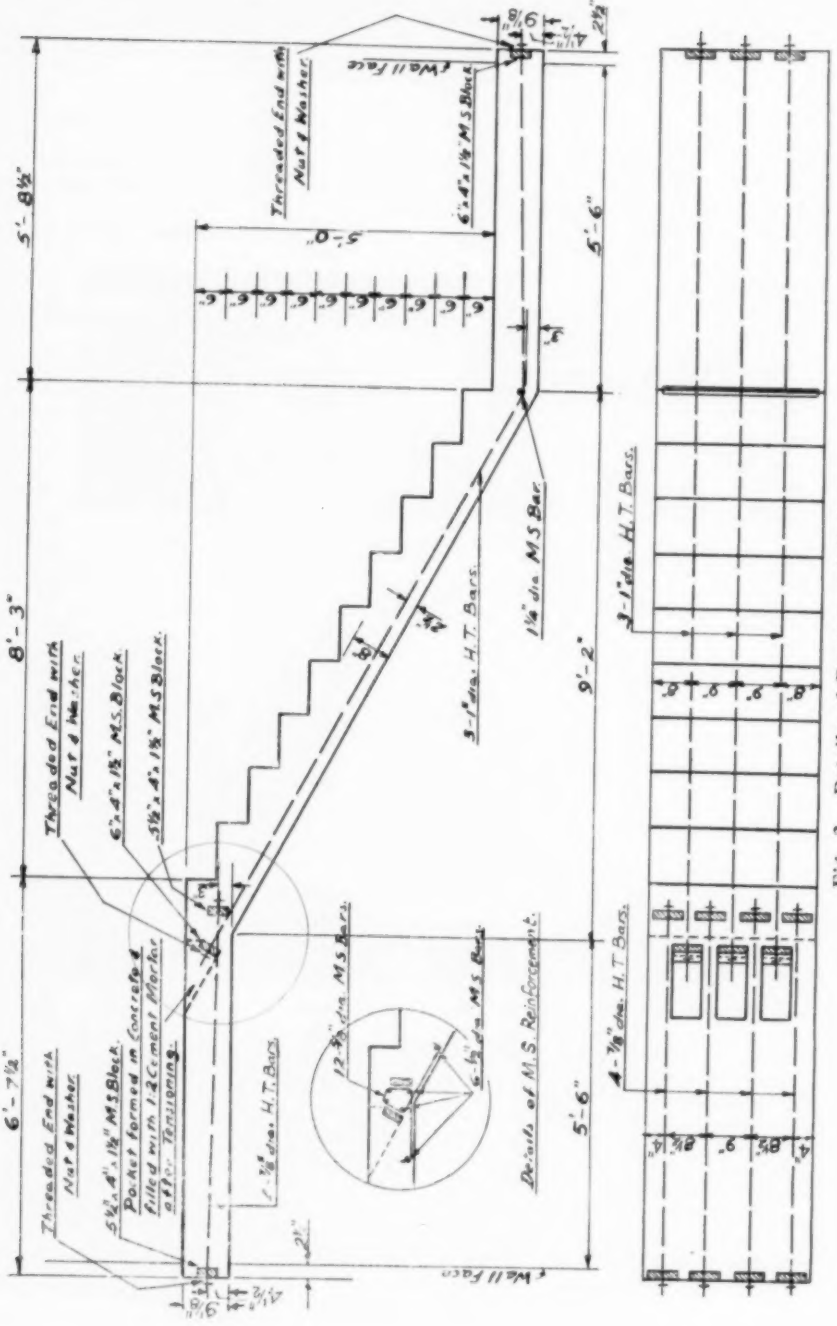


Fig. 3.—Details of Prestressed Staircase.

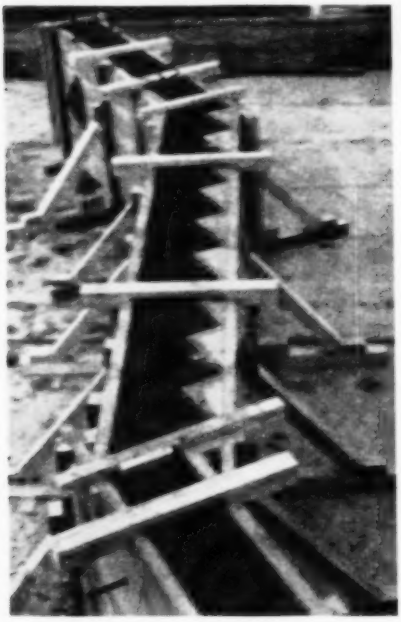


Fig. 4.—Timber Mould for a Prestressed Staircase.

The specification required a strength of 4500 lb. per square inch before tensioning, which was done fourteen days after casting. The 1-in. diameter bars were tensioned from each end, the crank being treated as a fixed point, although in fact this was not so because when one of the jacks only was operated the gauge on the other jack registered a corresponding increase of pull. The bars were tensioned to 42 tons per square inch and the actual extensions were checked against the calculated extensions.

One flight was tested to destruction over a clear span of 20 ft. 2 in.; *Fig.* 6 shows it being lifted on to the supports. The test was made forty-three days after casting. The deflections under light loads were recorded on two gauges, and three surveyor's levels were used to measure the deflections of scales fixed at the lower support, at midspan, and 2 ft. from the upper support. Concrete kerbs and sleepers were used for loading, and with a uniformly-distributed load of 8790 lb. in addition to its own weight, representing the weight of the finishes plus 1½ times the superimposed load, the soffit and edges were examined with a magnifying glass, but no sign of cracking was detected. The vertical deflections at midspan were 0·1021 in. and 0·235 in. at the right-hand and left-hand sides respectively. The load was then increased to 15,792 lb. and a crack was observed with a magnifying glass at about the middle of the span. At a total load of 26,880 lb. cracks appeared across the soffit at about 2 ft. intervals, each crack being about $\frac{1}{32}$ in. wide and appearing on the edges to a depth of about 5 in.; the deflection at the centre was about $\frac{7}{8}$ in. Ten of the sleepers were then moved from the landings to the stair flight, thus concentrating the load in the central portion. The deflection increased steadily, as each

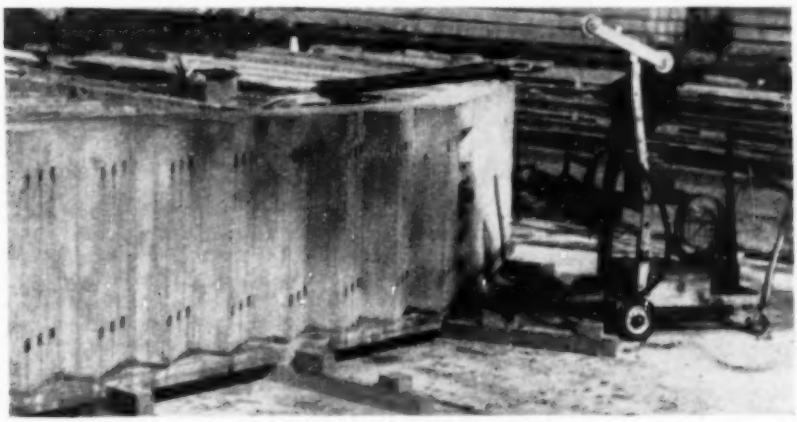


Fig. 5.—Tensioning the Bars.

sleeper was moved, to 1¾ in., and the cracks widened to about ¾ in. Upon moving one sleeper, at about mid-span, to form a seating for further loading, sudden failure occurred by crushing of the concrete at the third tread from the lower landing (*Fig.* 2). There was a large increase in the width of the crack in the soffit (*Fig.* 7), and the lower ends of the treads settled about 3 in. on to the staging (*Fig.* 2).

The total load at failure represents the weight of the stairs and landings, the weight of the finish, a superimposed load of 100 lb. per square foot, and 20,130 lb. spread over the central 9 ft. 2 in. of the span.

On removal of the load considerable recovery of the deflection occurred with partial closing of the main crack. *Fig.* 1 shows the flight after the removal of the load. The calculated stresses in the concrete at mid-span under the design load and the prestressing forces, after allowing for a loss of 16 per cent. due to creep and shrinkage, were 1148 lb. per square inch at the top of the waist and 223 lb. per square inch at the bottom of the waist. Under the load at failure the compressive stress in the concrete at the top of the waist was calculated to be 4080 lb. per square inch and the tensile stress in the steel 136,500 lb. per square inch.

Fig. 6.—Lifting the Staircase on to Supports.

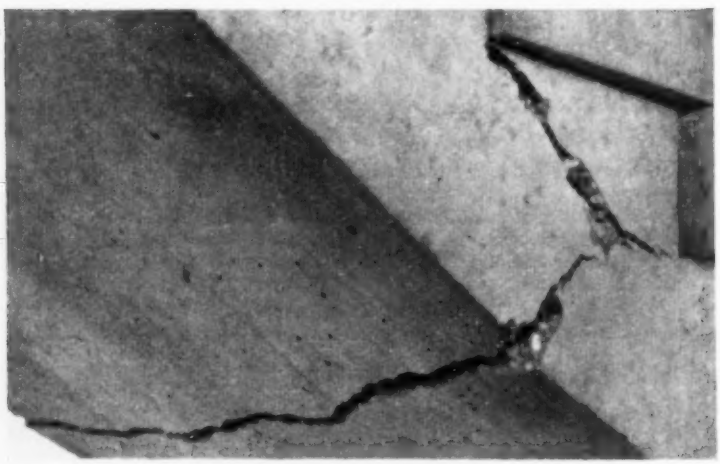


Fig. 7.—Crack at Failure.

Mr. George Whitby, M.B.E., A.R.I.B.A., is the architect for the school, and Mr. R. F. Galbraith, B.Sc., A.M.I.C.E., M.I.Struct.E., the consulting engineer who designed the staircase. Messrs. McCalls Macalloy, Ltd., supplied the high-tensile bars, fittings, and jacks, and Stent Precast Concrete, Ltd., manufactured the stairs and carried out the test, as sub-contractors to Messrs. Bernard Sunley & Sons, Ltd.

Multiple-Story Dwellings at Mitcham.

THE building shown in *Fig.* 1 is part of a large housing scheme at Pollards Hill, Mitcham, Surrey. In addition to four six-story buildings in which the dwellings are each on two stories, the scheme includes three four-story blocks of flats and precast concrete and brick two-story houses.

The buildings containing the maisonettes comprise a reinforced concrete frame, with external cavity walls of brick and internal walls of clinker blocks, supported on piles. Each block comprises three buildings, in the shape of a letter U, of which the base is about 190 ft. long and the arms 95 ft. long. The buildings accommodate three two-story maisonettes in their height. The width of the building is 24 ft. and the bedroom floors project 5 ft. beyond the face of the building over the access balconies on the second and fourth floors. *Fig.* 2 shows the method of construction and the sizes of the members, which are standardised throughout the work. Although a slab is provided at the ceiling level of the top story, a hipped roof was built to accommodate water-tanks and other fittings. As the slab supports the tanks and acts as the tie-member of the roof-truss, the construction is no more expensive than the alternatives of providing waterproof finishes to the slab and building parapet walls, or building stronger trusses which would carry the tanks. Also, the construction adopted provides thermal insulation to the top floor.

Bores showed that there is about 20 ft. of brown clay overlying blue clay, except in one part of the site where there is wet sand and gravel for a depth of about 13 ft., below which blue clay is again encountered. Tests of the brown clay indicated that the maximum bearing pressure should be only 1 ton per square foot and that considerable, and possibly differential, settlement might be expected. Consequently in-situ bored piles, with caps and tie-beams at ground level, were used. The buildings required 1303 piles, each with a nominal diameter of 1 ft. $5\frac{1}{2}$ in. and a working load of 50 tons. The average length of the piles is about 26 ft., which allows a penetration of not less than 6 ft. into the blue clay. The tie-beams support the ground-floor slab. A 2-in. layer of concrete was placed on the ground under all tie-beams and pile caps, and the shutters for the sides of the beams were formed of brickwork.

Fig. 1.—The Six-Story Blocks of Dwellings.

Construction.

The methods of construction were largely determined by the use of a derrick crane with a 100-ft. jib and, to avoid having to move the crane several times

during the building of each block, a construction joint was made in the centre of the building forming the base of the U-shape. From one position the crane could then be used for a length of about 200 ft., each half of the building being constructed separately (*Figs.* 3 and 4).

The concrete was a nominal 1 : 2 : 4 mixture, the sand and coarse aggregates being weigh-batched in a plant supplied by a belt-conveyor. The average slump of the concrete was 2½ in. and average crushing strengths of 3220 lb. per square inch at seven days and 5477 lb. per square inch at 28 days were obtained. The concrete was taken directly from the mixer in buckets by the crane, hoisted, and placed in position.

All the shuttering was made on the site. For the slabs a telescopic type of shuttering was used, supported by the sides of the beam shutters and props in the centre of the span. Each unit covered an area of about 1 sq. yd. The shutters for the beams were of 1¼-in. timber, with 3-in. by 4-in. timbers placed underneath and supporting the beam soffits at about 5-ft. intervals. These timbers, supported by props, passed from the front to the rear of the building and projected on each side about 1 ft. 9 in. On these projections a platform was made all around the building, thus eliminating the need for external scaffolding. In moving the shuttering from one floor to the next the crane was used to pick up the shutters from platforms formed by two of the telescopic shutters cantilevered from the face of the building and held

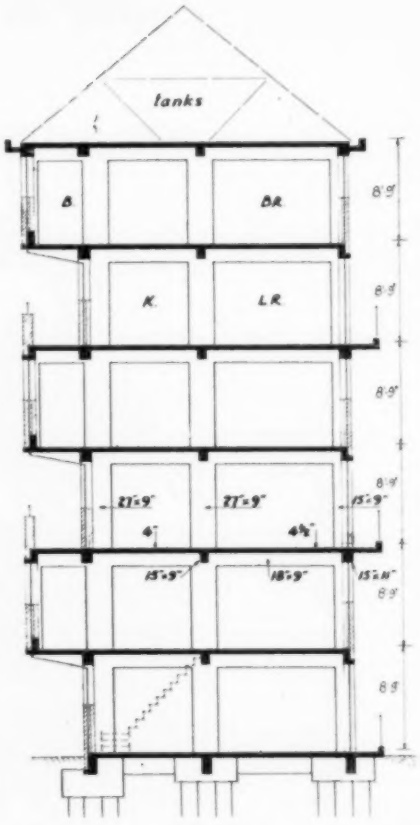


Fig. 2.—Cross Section through the Maisonettes.

Fig. 3.—Construction of the Frame for Half of a Block.

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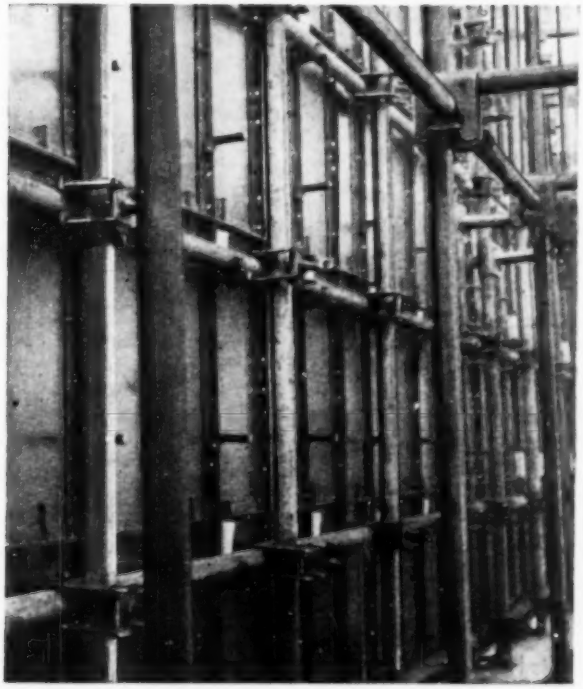

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Fig. 4.—Construction of Each Half-Block being Carried Out Separately.

down by steel props wedged between the floor and the ceiling.

On the first block the reinforced concrete work occupied 20 weeks, on the second 16 weeks, on the third 15 weeks, and on the fourth 14 weeks. The average number of men employed on the concrete work was 40, comprising 16 carpenters, 8 carpenters' labourers, 8 steel-fixers, 7 labourers, and a crane driver. The total time required for building each block averaged 11½ months.

The architects appointed by the Corporation of Mitcham, whose Engineer and Surveyor is Mr. Riley Schofield, A.M.I.C.E., were Messrs. Collcutt & Hamp. The structural engineers were Messrs. John Liversedge & Associates, and the contractors Messrs. Wates, Ltd. The soil investigations were carried out by Messrs. Le Grand, Sutcliffe & Gell; piling was by the Cementation Co., Ltd.

A New Soil-Boring Machine.

A NEW soil-boring machine which can be mounted on a lorry or trailer has been developed at the Road Research Laboratory. The machine can drill clean holes up to 20 in. diameter and 16 ft. deep in most soils. The boring-bar can be brought from the travelling to the working position in one minute and it will bore vertical or inclined holes at a rate of about 1 ft. a minute. The machine can also be used for boring holes for short piles; the piles are formed by filling the holes with concrete.

The machine is powered by a 7-h.p. diesel engine. The chief feature is a hollow lattice mast 9 in. square, which is raised from its sloping travelling position to a vertical position by means of two screws controlled by a friction clutch. The boring-bar passes through a square hole in a gear-wheel controlled by a clutch. The driving shafts for rotating and lifting the bar are arranged co-axially with the main pivots to permit boring with the mast inclined. Three cutting tools are used, namely one for fine soils, one for soils containing small stones, and one for soils with larger stones.

The late Franz Dischinger.

THE death is announced of Professor Dr. Ing. Franz Dischinger, the well-known German authority on reinforced concrete, at the age of sixty-six years.

Circular Reinforced Concrete Hangars.

Two aeroplane hangars of a new type recently constructed in Belgium are described in "La Technique des Travaux" for January-February 1952. Each can accommodate aeroplanes on an area of 2500 sq. yd. (*Fig.* 1).

The roof is a circular concrete shell with an overall diameter of 164 ft. and a maximum height of 23 ft. It consists of a central dome of 72 ft. diameter with an outer surrounding shell cantilevered a distance of 46 ft. The whole is supported on four columns placed at the intersection of the dome and the cantilevered shell. A cross section is given in *Fig.* 2 and a plan in *Fig.* 3. The thickness of the roof varies from 3 in. in the dome to $4\frac{3}{4}$ in. in the outer shell. The reinforcement comprises concentric rings which resist tensile forces and radial bars to resist compressive forces, as shown in *Figs.* 4 and 5. Concreting was carried out continuously by casting one complete ring per day, ten days being taken to complete the superstructure.

The hangar is closed by means of sliding metal doors traversing the circumference of 490 ft. The doors slide on two rails which make it possible to open half the hangar in any direction.

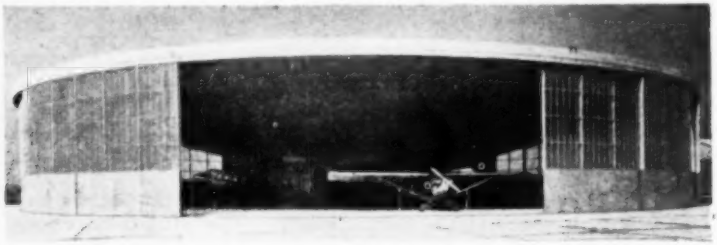

Fig. 1.

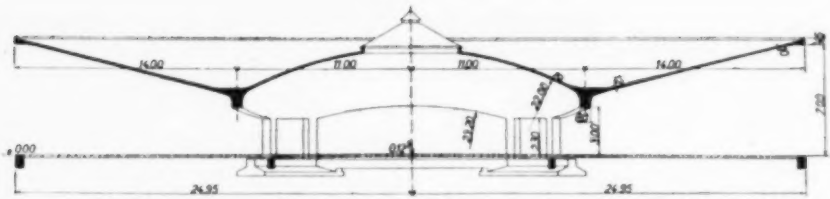

Fig. 2.—Cross Section through Hangar.

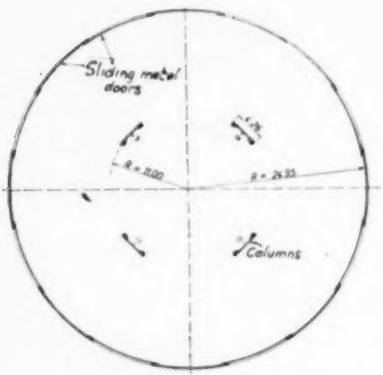

Fig. 3.—Plan of Hangar.

The article gives a brief abstract of the calculations, which are similar to those for a reservoir to contain water. The shape of the structure eliminates bending, and allows a very light form of roof. There are no external walls, their place being taken by the sliding doors. The construction consists of a central skeleton with a self-supporting shell forming the roof which is monolithic with the supporting columns. There are no tie-beams or diaphragms to stiffen the roof. At the junction of the dome and the outer shell

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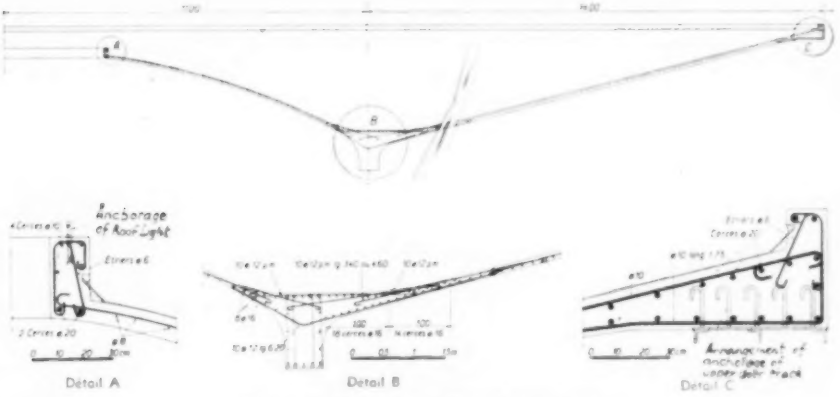


Fig. 4.—Details of Reinforcement.

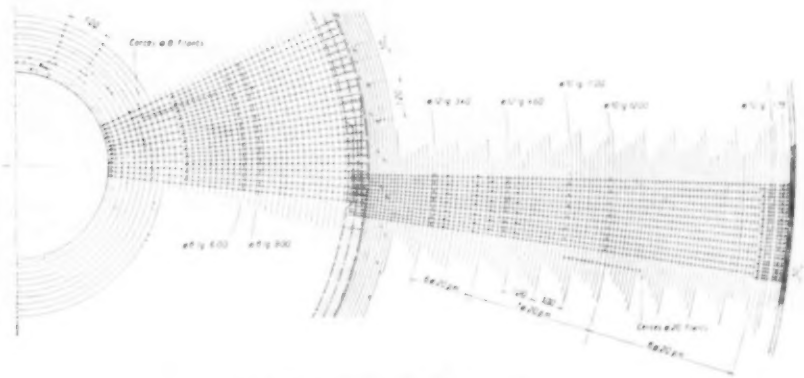


Fig. 5.—Plan of Reinforcement.

is a ring-beam spanning from column to column. This beam carries a heavy compressive load in addition to the bending moments and torsions. The central dome is in compression only and the maximum stress is about 60 lb. per square inch. The outer shell is designed to carry the weight of the sliding doors in addition to the normal superimposed load.

The central portion of the hangar, which may be used as a workshop, is lit by a circular roof-light. The roof may be used as a reservoir or a swimming pool, the columns having been designed for the extra load.

Standard Method of Measurement of Prestressed Concrete.

A SUPPLEMENT to the "Standard Method of Measurement of Building Works" dealing with prestressed concrete has recently been published by the Royal Institution of Chartered Surveyors and the National Federation of Building Trades Employers. The method of measurement set out in the supplement was drawn up with the collaboration of the Prestressed Concrete Group of the Cement and Concrete Association. Copies (price 1s.) may be obtained from the Institution or the Federation.

Research on Concrete.

THE following are abstracts from the report for the year 1951 of the Building Research Board, which was published recently by H.M. Stationery Office (price 4*s.* 6*d.*), and describe some of the investigations completed or in progress.

Reinforced Concrete Walls.

To provide information on the contribution of the reinforcement to the strength of reinforced concrete walls, half full-size models reinforced with two layers of steel were loaded in axial and eccentric compression. The minimum requirements of 0·4 per cent. vertical steel and 0·2 per cent. horizontal steel of the British Standard Code of Practice No. 114 were satisfied in the walls, and the load was applied between hinges in a manner similar to that adopted in previous tests on unreinforced models. Initial eccentricities of load up to one-third of the thickness of the wall were investigated, and measurements were made of the strains in the concrete and steel and the lateral deflections of the walls under load. The strains in the steel were measured using electrical-resistance strain-gauges protected by waterproof covers devised for use on steel to be encased in concrete.

The yield strength of both layers of steel was developed at failure of the axially loaded walls, and the ultimate axial strength exceeded that of a similar wall without reinforcement by an amount not less than the yield strength of the steel. The ultimate stress in the concrete was a greater proportion of the cube strength than in previous tests on similar unreinforced walls, but further tests are needed before a general conclusion will be justified. Under eccentric loading the contribution of the reinforcement to the strength of the wall was not less than the yield strength of one of the two layers of steel. Stresses approaching the yield point were measured in the compression steel with initial eccentricities up to one-sixth of the thickness of the wall, whereas similar stresses were measured in the tensile steel with greater initial eccentricities up to one-third of the thickness.

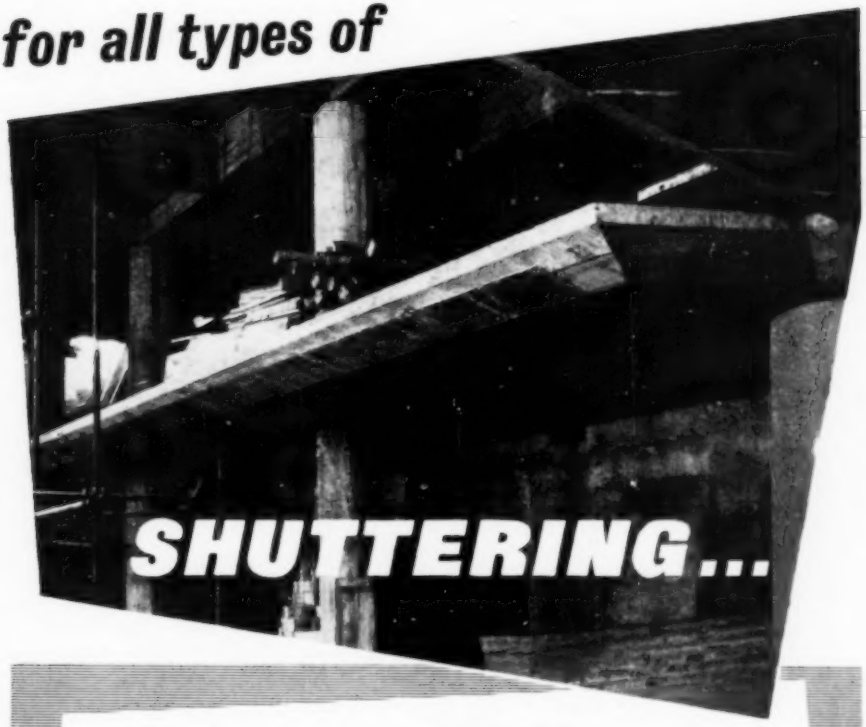
Prestressed Concrete.

The examination of the properties of hard-drawn steel wire has been extended to include wires in coils of large diameter. Particular attention has been directed to the behaviour of wire under sustained loading. For all the samples of wire tested, the creep has been sufficient to require consideration in estimating the loss of prestressing force. Experiments are being made to determine whether the amount of creep can be assessed from tests of short duration. The suitability of this type of test cannot be gauged until the present tests of long duration are complete, but for some samples of wire a reasonably accurate assessment of the creep over a period of several months has been made from tests of 30 minutes' duration. Temporary over-stressing of the wire in the tensioning operation has been advocated as a means of reducing the loss due to creep; this has been found effective in reducing the rate of creep during the first few days. Creep is also reduced if the rate of stressing during tensioning is reduced.

The investigation of the resistance to impact of prestressed concrete beams with pretensioned steel has indicated that the energy absorbed in low-velocity impact tests can be related to the energy of deformation in a static test, provided that the form of failure is the same in each test. It has also been shown that there may be a critical proportion of steel in the section for greatest resistance to impact, and that this proportion may be substantially influenced by the strength of the concrete. An investigation of the influence of the span on the total energy of deformation of similar sections in static tests has shown that the energy is almost independent of the span, which suggests that the greater part of the energy is absorbed in the region of the fracture of the beam.

Several systems of construction for domestic floors have been submitted to the Joint Fire Research Organization for standard fire tests. These have shown that for house construction, where a fire resistance of only half an hour is required, adequate protection to the prestressed concrete beams can easily be provided. An investigation of the behaviour of composite prestressed concrete beams with post-tensioned cables has been undertaken by the Joint Fire Research Organ-

for all types of



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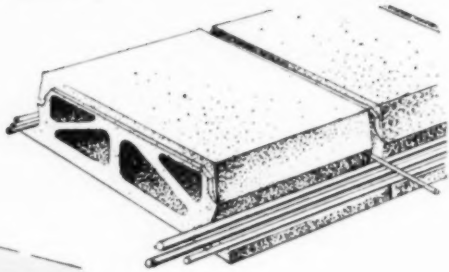
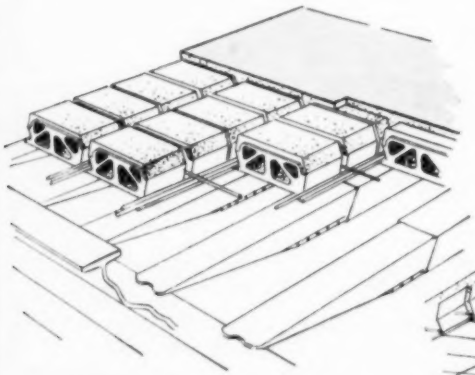
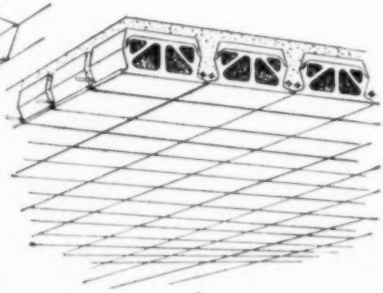




Showing Two-way Reinforcement and Hollow Concrete Blocks laid on Trianco Telescopic Centers.



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ization with the collaboration of the Building Research Station. Cracking of precast concrete products during steam curing at atmospheric pressure on "long-line" pretensioning beds has been found to be caused by deferring the release of the force in the wires until the concrete has cooled, and can be avoided by releasing the force immediately on conclusion of the steam-curing treatment. Tests on 5½-in. by 2-in. prestressed joists have shown that 12 S.W.G. wires can be replaced satisfactorily by one-third the number of wires of 0·2 in. diameter, thus simplifying manufacture. Developments of equipment have included a plastic end for moulds on pre-tensioning beds which can be inserted after the wires are anchored, and work is proceeding on the design of apparatus for the direct measurement of the stress in the wires during the tensioning operation which will eliminate the errors that may occur if the stress in the steel is gauged from the extension of the wires or the pressure in jacks.

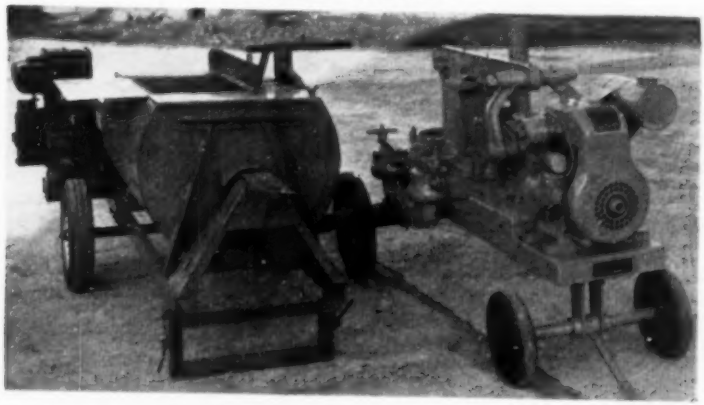
Concrete-encased Steelwork.

The stiffening and strengthening effects of the concrete used for encasing structural steel for fire protection are under investigation with a view to economy in the use of structural steel in buildings. The effects of various thicknesses of encasement were measured in tests on simply-supported 6-in. by 3-in. rolled steel joists. The central deflection at working loads of a joist with 1 in. cover of concrete with a cube strength about 4000 lb. per square inch was 80 per cent. of that of a similar joist without encasement, whilst the ultimate load was 15 per cent. greater than that of the bare joist. When the cover was increased to 2 in. the corresponding deflection ratio diminished to 50 per cent. and the ultimate load increased to 50 per cent. more than that of the joist.

Vibrations in Structures.

A considerable number of measurements of vibration amplitude and frequency were made during the year to investigate the vibration produced in houses by the operation of aircraft at London Airport. The results show that the vibration amplitudes were usually 0·0001 in. or less.

Grout Mixers.



THE illustration shows a new type of batch grout mixer and pump. The mixer is available in sizes from a hand-operated machine with a capacity per batch of 1 cwt. of cement, 2½ cu. ft. of sand, and 7 to 8 gallons of water to a motor-driven machine with a capacity per batch of 4 cwt. of cement, 10 cu. ft. of sand, and 28 to 32 gallons of water. The grout may be discharged from the mixer by gravity or, as shown in the illustration, by a pump. In the latter case the grout may be conveyed up to 180 ft. with a single-diaphragm pump and up to 300 ft. with a double-diaphragm pump. The output per hour of the largest mixer is sufficient for grouting about 140 sq. yd. of slab 6 in. thick. The mixers and pumps are supplied by Cheecol Processes Ltd., of Reading.

Thermic Boring through Reinforced Concrete.

In drilling holes in reinforced concrete difficulty is always experienced in cutting through reinforcement, and in many cases the vibration and noise which accompany drilling prohibit its use. Thermic boring is a method of boring through reinforced concrete, stone or similar materials by the fusion of oxygen and iron, the temperature of which is sufficient to melt the material. In the case of concrete the silica content becomes fluid and is carried away with the products of combustion, the presence of steel actually facilitating the process and the work being carried out without noise or vibration.

The equipment consists of three steel containers for oxygen connected by a manifold with pressure gauges from which the oxygen passes through flexible hoses to a holder into which is inserted a mild steel tube filled with a number of soft-iron wires. The size of the tube and the number of wires are determined by the size of the hole required.

The point of the tube is first heated to a red heat by an oxy-acetylene burner or some other method and oxygen is fed through it. The end of the tube then fuses and is applied to the material to be penetrated. The heat generated melts the material and the flow of slag is assisted by the molten metal. Holes can be bored horizontally or vertically up to about 15 ft. to 20 ft. long, but in boring vertically difficulty is experienced beyond a depth of about 10 ft. after which a special system for removing the slag is required.

The diameter of the hole produced can be varied by using tubes of different diameters, one of $\frac{3}{8}$-in. producing a hole between $2\frac{1}{2}$ in. and 3 in. diameter. The speed of penetration for a horizontal hole of this size is about $1\frac{1}{2}$ minutes per foot, and six to seven feet of the tube and wires and about 23 cu. ft. of oxygen are consumed, but this depends largely upon the silica content of the material being bored. Larger holes, up to 4 in. diameter, can be produced by tubes up to 1 in. diameter. For vertical boring the consumption of the tube and wires and of oxygen is the same as with horizontal boring for a depth of about 8 ft., beyond which it increases to about 8 ft. to 10 ft. of tube and 30 to 32 cu. ft. of oxygen per foot of penetration.

Although the method is for boring and not for cutting it is possible to cut concrete by boring a series of closely spaced holes along the line of the cut required. The system, which is operated by Messrs. M. Semet & Co., Ltd., has been used in this way to cut holes through reinforced concrete foundation beams 3 ft. 9 in. thick immediately behind which was 1 ft. $1\frac{1}{2}$ in. of brickwork. One of the holes, 1 ft. 2 in. by 2 ft. 2 in., required the boring of 20 holes, consuming 2970 cu. ft. of oxygen and 462 ft. of $\frac{1}{2}$-in. tube packed with nine soft-iron wires; the total weight of iron and steel used was 577 lb. The work took 13 hours. Another application required the cutting of three holes, 4 ft. diameter by 1 ft. thick, in a reinforced concrete roof at the offices of Messrs. Charles R. Price. The circles were marked-out by cutting a chase 1 in. wide by 1 in. deep with a $\frac{1}{2}$-in. tube and then boring holes about 2 in. apart around the circumference, using a steel sheet for protection. The sheet was then removed and the portions between the holes burnt away, the slabs having previously been attached to a block and tackle. About 150 holes were bored requiring 1331 ft. of $\frac{1}{2}$-in. tube and 15,400 cu. ft. of oxygen. The work was carried out by three men in 90 hours.

A further application of the system for boring holes through a concrete structure to form ducts for prestressing cables was described on page 11 of this journal for January, 1953.

Design of Rigid-Frame Bridges.

In the article by Mr. Andrew Gallia on the "Design of Rigid-Frame Bridges with Kerbs" in the December, 1952, number of this journal, the expression for the stiffness-factor of the kerb and deck slab given on page 363 should be

$$H = \frac{b_1(1-\mu^2)}{a}\left(\frac{h_1}{h}\right)^3,$$

and in the example on page 370 the numerical value of H should be

$$\frac{18}{23{\cdot}5 \times 12}(1-0{\cdot}15^2)\left(\frac{28{\cdot}5}{14{\cdot}5}\right)^3.$$

MISCELLANEOUS ADVERTISEMENTS.

Situations Wanted, 3d. a word : minimum 7s. 6d. Situations Vacant, 4d. a word : minimum 10s. Other miscellaneous advertisements, 4d. a word : 10s. minimum. Box number 1s. extra. The engagement of persons answering these advertisements is subject to the Notification of Vacancies Order, 1952.

Advertisements must reach this office by the 23rd of the month preceding publication.

SITUATIONS VACANT.

SITUATIONS VACANT. MINISTRY OF WORKS.—Structural engineering draughtsmen required in London, at Risley (Warrington), Harwell (near Didcot, Berks.), and other centres. Accommodation available for single men at Risley and Harwell. London salaries—Draughtsmen : Up to £628 per annum. Starting pay according to age, qualifications, and experience. Leading draughtsmen : £623 to £733 per annum. Rates outside London slightly lower. Applicants should be experienced in design and/or detailing of reinforced concrete or structural steelwork. Work is varied and not confined to standard schemes. Although not established posts, many have long-term possibilities : competitions are held periodically to fill established vacancies. Apply in writing stating age, nationality, experience, and locality preferred to CHIEF STRUCTURAL ENGINEER, W.G. 10/S.1, Ministry of Works, Abell House, John Islip Street, London, S.W.1.

SITUATION VACANT. Draughtsman (senior) required in London drawing office. Experience in layout work and the application of steel shuttering as used in the building and civil engineering trade would be an advantage. Excellent prospects of promotion at home or abroad within this rapidly-expanding Company. Write stating experience, age, and salary required. ACROW (ENGINEERS), LTD., South Wharf, London, W.2. Reference (JRT) Engineering Department.

SITUATIONS VACANT. Experienced reinforced concrete designers required for Bristol and Sunbury-on-Thames offices of reinforcement manufacturers. Positions offer opportunity to gain experience of wide range of structures. Attractive salaries offered to suitable applicants. Applications in confidence, stating qualifications and fullest details of experience, to BOX 2609, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Young qualified detailer-designer with previous experience of reinforced concrete required in Birmingham. Some additional experience of structural steelwork and quantities an advantage. Write, giving full particulars of age, education, qualifications, experience, and salary required, to HENRY M. HALE & PARTNERS, CHARTERED STRUCTURAL ENGINEERS, 125 Edmund Street, Birmingham, 3.

SITUATIONS VACANT. Reinforced concrete designers and detailers required immediately. Knowledge of colliery structures desired but not essential. Experience of quantities an advantage. Staff pension and bonus schemes in operation. BOX 3632, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATIONS VACANT. Designer-draughtsmen required for London office. Experience in reinforced concrete frames, floors, roof and staircase construction essential. Progressive post. Pension scheme. Alternate Saturdays free. Write fully experience and salary required to "O." DIESPEKER & Co., LTD., Clifton House, Euston Road, London, N.W.1.

SITUATIONS VACANT. General civil and structural designer-draughtsmen are required by Imperial Chemical Industries Limited, General Chemicals Division, to assist in the design of steel and reinforced concrete structures for chemical plants. Location : Runcorn. Salary dependent on age and experience. Apply in writing, quoting E/83, to STAFF MANAGER, IMPERIAL CHEMICAL INDUSTRIES LIMITED, General Chemicals Division, Cunard Building, Liverpool, 3.

SITUATION VACANT. Detailing draughtsman required for precast concrete manufacturers, Westminster office. Should have completed National Service. National certificate an advantage. 5-days' week. Write BOX 55, WALTER JUDD, LTD., 47 Gresham Street, London, E.C.2.

SITUATION VACANT. Old-established Tees-Side firm requires section leader reinforced concrete designer-draughtsman fully experienced in designing and detailing reinforced concrete structures, foundations, and other civil work. Apply, giving full particulars and experience, quoting reference "D," to BOX 3638, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Consulting structural engineer, Westminster, requires senior designer-draughtsman with first-class experience in reinforced concrete for responsible position. Experience in structural steelwork an advantage. High salary and good prospects for suitable applicant. Write in confidence stating age, qualifications, and full details of experience. BOX 3639, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. The Trussed Concrete Steel Co., Ltd., Truscon House, 35–41 Lower Marsh, London, S.E.1, have a vacancy in their London office for a measurer with considerable experience in the taking-off and preparation of bills of quantities mainly for reinforced concrete structures. Age 25–30. Five-days' week, and pension scheme. Apply in writing to the above address, giving full particulars of age, education, and previous employment.

SITUATIONS VACANT. The Trussed Concrete Steel Co., Ltd., Truscon House, 35–41 Lower Marsh, London, S.E.1, have vacancies in their London office for 3 designers and 5 designer-detailers with considerable experience in reinforced concrete D.O. work. Five-days' week, and pension scheme. Apply in writing to the above address, giving full particulars of age, education, and previous employment.

SITUATION VACANT. Specialist firm of reinforced concrete designer/contractors require the services of an experienced contracts supervisor. The appointment calls for an energetic man, preferably, but not essentially, qualified, having sound technical knowledge and good administrative capabilities. Applications will be treated in strict confidence and should give details of training, experience, present interest in the development of the industry, and salary required. Present staff have been advised of this advertisement. BOX 3641, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Designer, with sound knowledge and experience of designing and detailing reinforced concrete structures, required for Regional Headquarters Office, London. Commencing salary £612 5s. od. per annum. Certain residential and other travelling facilities provided. Applications, stating age, qualifications, and experience, to CIVIL ENGINEER, LONDON MIDLAND REGION, BRITISH RAILWAYS, Euston Grove, London, N.W.1.

SITUATION VACANT. Prestressed concrete floors. Private company in Mid-Essex specialising in prestressed construction requires assistant to managing director. The applicant may be either a junior with some training or a fully experienced and competent person who is suitable to qualify for a directorship. Please write fully to BOX 3640, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Civil engineer age 30/35 required. Must have several years' experience outside with contractors on piling, foundations, and reinforced concrete. Knowledge of jetty construction an advantage. Write brief details, qualifications, training, and experience to J. L. KIER & Co., LTD., 7 Lygon Place, Westminster, S.W.1.

SITUATIONS VACANT. The British Reinforced Concrete Engineering Co., Ltd., require several qualified designers and detailers with specialist experience for their Stafford, London, Newcastle-upon-Tyne, and Dublin offices. Five-days' week and staff pension scheme. Apply to B.R.C. ENGINEERING CO., LTD., Stafford.

SITUATION VACANT. Chief assistant required for consulting engineer's office, Westminster, to supervise design office, visit sites, and carry out administrative duties, etc. First-class knowledge of reinforced concrete practice essential. Good commencing salary according to experience. This appointment offers good prospects to suitable candidate. Apply, giving full particulars, to BOX 3643, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

(*Continued on p. lxiv*).

(*Continued from p. lxiii*)

SITUATION WANTED. Professional civil and structural engineer (32) seeks senior position in design office. Extensive experience in reinforced and prestressed concrete, soil mechanics, and foundations. Knowledge of concrete shell design. Several years with first-class consultants. Box 3642, Concrete and Constructional Engineering, 14 Dartmouth Street, London, S.W.1.

PROFESSIONAL SERVICES.

PROFESSIONAL SERVICES. Reinforced concrete designer-draughtsman with 20 years' experience in the design of reinforced concrete structures, foundations for heavy industrial structures, chemical plant, and oil refineries, undertakes to prepare complete designs, calculations, and working details to help consulting engineers. Excellent references, moderate fee. Box 3637, Concrete and Constructional Engineering, 14 Dartmouth Street, London, S.W.1.

FOR SALE.

FOR SALE. Sacks, bags, and curing cloths for sale. You want the best type and quickest delivery. Write John Braydon, Ltd., 26 The Highway, London, E.1. Telephone: ROYal 1044.

FOR SALE. Steel angle fencing stakes. All sizes in stock. List on request. E. Stephens & Son, Ltd., Bath Street, London, E.C.1.

FOR SALE. Steel angle pegs, 24 in. × 1½ in. by ⅜ in., also 12 in. × ¼ in. Cheap price for quantities. E. Stephens & Son, Ltd., Bath Street, London, E.C.1.

FOR SALE. Heavy (tinned inside) black japanned outside, cylinder shape, plan and map cases, 3 ft. 2 in. long, 5½ in. diameter, fitted with carrying handle, hinged lid with clasp for locking up, weight 11 lb. Excellent for storing plans, maps, etc. 12s. each, carriage paid. Excellent value. Whites, Calvert Street, Norwich.

FOR SALE. Ducting suitable for land drainage, piling, etc. 6½ in. O/D × 14G. × 18 ft. long, at 2s. per foot. Discount for quantities. Stock list on application. E. Stephens & Son, Bath Street, London, E.C.1.

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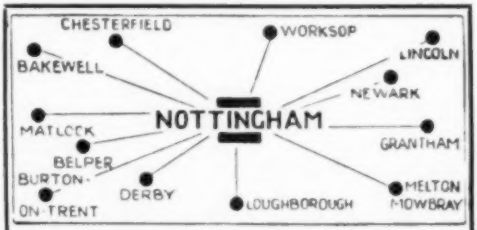
FOR HIRE. Lattice steel erection masts (light and heavy), 30 ft. to 150 ft. high, for immediate hire. Bellman's, Terminal House, London, S.W.1. Telephone: Sloane 5259.



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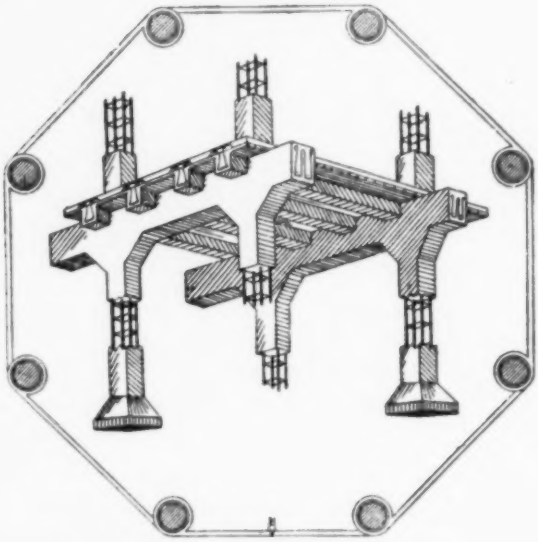

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